Structural and Economic Aspects of the use of semi-rigid Joints in Steel Frames.

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To my father, my mother, my wife and my children.

Contents

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Declaration

I declare that the work presented in this thesis was carried out by the author and has not been taken from any other research except with specific reference has been made. None of this work has been submitted for a higher degree at any other establishment.

Summary

This thesis reports on five main areas as follows:

1. Braced steel frames designed for semi-continuous construction were studied to determine savings in both cost and weight. Various frame parameters such as the number of bays, use of grade S355 steel, beam spans, types of connection, and selection of beam size were investigated. The investigation confirmed that semi-continuous construction contributes to worthwhile percentage savings on both cost and weight.

2. Analysis and design of steel unbraced frames bending on both axes were performed with emphasis on stability and deflection checks. Rules are proposed to improve the stability and stiffness. For connections to the minor axis, a proposed joint detail is presented. The performance of the frames was checked for collapse load level at ULS; deflection limits at SLS were also checked; in both cases using first and second order analysis. The investigation demonstrated that the frames should be restricted to less than four storeys.

3. A study on minor axis joints was carried out for flush end plate connections connected to the column web. Previous experimental results of moment and stiffness were compared with predicted values. Moment values were predicted using Gomes' formulae. The stiffness due to the column web was predicted using finite element analysis. The results showed good agreement between experimental and predicted values. The study on the connections was extended to their suitability in steel frames bending about the minor axis; the investigation confirmed that the connections were not suitable for unbraced wind-moment frames. An equation for prediction of initial stiffness was nevertheless established for the connection.

4. Steel frames with composite beams designed for minimum wind combined with maximum gravity load were studied for their performance, taking into account cracking along the beams. The investigation showed that the frames meet the requirements of deflection and sustain a load level of 1.0 for ULS. For frames studied for maximum wind combined with minimum gravity load, the moment capacity of the joints governed the design which resulted in a deeper beam section.

5. Seven tests were carried out for a new type of shear connector system installed by compressed air. The aim of the tests was to study the shear capacity and ductility of the studs. The tests showed that the pins fail due to fracture and the stud systems needs some improvements to increase the key structural properties.

Notation

a_j	Dimensionless exponent which indicates the effect of jth size parameter on the M-Φ relationship.
A_s	Tensile stress area of the bolt or area of steel beam.
A	Area of the external surface of the building.
A_g	Gross cross-sectional area.
a_j	Exponent value for K factor.
B	Total width of the frame.
Be	Effective width length of composite beam.
C_1 and C_2	Constant values for linear equation.
C_{pe}	Pressure coefficient for the external face of the building.
C_{pi}	Pressure coefficient for the internal face of the building.
d	Shank diameter.
D	Depth of the steel beam.
D_s	Depth of the concrete flange.
D_p	Depth of the profile steel sheet.
E	Young's modulus for steel.
E_{1t}	Vertical distance between the tensile bolts an the outermost surface of the top flange of the beam.
E_a	Modulus of elasticity of steel.
E'_c	Effective modulus of concrete.
E_{cm}	Modulus of elasticity of concrete.

f_{cu}	Cube strength of concrete.
f_{ck}	Cylinder strength of concrete.
f_u	Ultimate tensile strength of the stud.
F	Applied axial load in member or wind force on a surface.
F_1 and F_2	Tension bolt forces.
F_{eq}.	Force of one bolt.
G	Horizontal distance between bolt centres.
$I_{1,2}$	Second moment of area of the upper beams in the storey.
$I_{2,2}$	Second moment of area of the lower beams in the storey.
$I_{3,2}$	Second moment of area of the internal designed column in the storey.
$I_{3,2'}$	Second moment of area of the external designed column in the storey.
I_g	Second moment of area of the uncracked section.
I_x	Second moment area of steel section.
h_1, h_2, h_3	Height of columns.
h_1	Lever arm of the connection measured from the first tension bolt row to the centre of compression in the beam flange.
h_2	Lever arm of the connection measured from the second tension bolt row to the centre of compression in the beam flange.
h_r	Distance between bolt row *r* and the centre of compression.
$k_{eff,r}$	Effective stiffness coefficient for a bolt row *r*.
k eq.	Equivalent stiffness coefficient for bolt row.
k_i	Stiffness coefficient representing deformation in the connection.

k	Degree of shear connectors.
k_5	Stiffness coefficients for the end plate.
k_7	Stiffness coefficients for bolts.
K	Standardisation factor.
$K_{j,ini}$	Joint initial secant stiffness.
K_j	Shear flexibility of the connection.
L_1, and L_2	Span of the beam.
L_b	Elongation length of the bolt.
L_f	Leg length of the flange weld.
L_w	Leg length of the web weld.
l_a	Connection lever arm.
l_{eff}	Length of a Tee-stub equivalent to the real pattern of yield lines occur around the bolt positions in the end plate.
m_{cf}	Horizontal distance between the bolt position and the column web.
m	Total number of size parameters.
m_{cp1}	Horizontal distance between the bolt position and the beam web.
m_{cp2}	Vertical distance between the underside of the beam's tension flange and the tensile bolts.
m	Equivalent moment acting on the member.
m_x and m_y	Equivalent uniform moment factor.
M	Moment applied to the connection.
$\overline{M}$	Equivalent uniform moment.

M_A	Maximum moment acting on the member
M_b	Lateral torsional buckling resistance moment.
M_c	Moment capacity of the connection or of a composite beam.
M_{cx}	Moment capacity about the major axis in the absence of axial load.
$M_{j,Rd}$	Design moment resistance of the connection.
$M_{hogging}$	Hogging moment.
M_s	Moment capacity of steel section.
M_x	Applied moment about the major axis.
M_y	Applied moment about the minor axis.
n	Slenderness correction factor.
p_b	Bending strength.
p_c	Compressive strength.
p_j	Numerical value of jth size parameter.
p_y	Design strength.
P_1, P_2, and P_3	Total horizontal shear in the column of the storey being designed.
P_u	Ultimate resistance per stud.
q	Dynamic pressure from design wind speed.
R_c	Total resistance of concrete flange.
R_q	Total resistance of the shear connectors.
S_1	Topography factor.
S_2	Ground roughness factor.
S_3	Statistical concepts factor.

S_j	Rotational stiffness of a joint.
$S_{j,ini}$	Initial rotational stiffness of the connection.
S_x	Plastic modulus of the section about the major axis.
r_y	Radius of gyration about the minor axis of the member.
t_p and t_{cp}	Thickness of the end plate.
t_{wb}	Thickness of the beam's web.
t_{cf}	Thickness of the column flange.
t_{bf}	Thickness of the beam flange.
t_{cw}	Thickness of the column web.
u	Buckling parameter.
v	Slenderness factor.
V	Basic wind speed.
V_s	Design wind speed.
W_1 and W_2	Cost factor.
W_3	Cost optimization factor.
y_c	Distance from neutral axis to the top surface of concrete.
z	Lever arm of the connection.
Z_y	Elastic section modulus about the minor axis.

Greek letters

μ	Stiffness ratio.
λ	Slenderness with effective length divided by the radius of gyration.
λ_{LT}	Equivalent slenderness.
Δ	Allowable sway over the storey height.
Φ	Rotational deformation of connection in radians.
α_e	Modulus ratio for (E_a/E'_c).
σ_{ct}	Maximum tensile stress.
β	Bending moment distribution factor.
δ	Displacement.

Chapter 1

1.0 Introduction.

1.1 Steel frames.

A steel frame for a building comprises members such as beams and columns, joined together by connections and designed to act together to resist load. The types of joints between the members play an important part in the behaviour of the frame under load and strongly influence the final cost of the structural system. It has been reported that although the connections may only account for roughly 5% of the total steel weight, they account for 40% of the labour cost[1.1]. In design methods conventionally used in practice, joints are categorised as either "pinned" or "rigid". However, for most joints a more appropriate classification of behaviour is as "semi-rigid"(see Figure 1.1) or "partially restrained"[1.2], resulting in what EC3[1.3] terms "semi-continuous" construction.

This thesis concerns both economic and structural aspects of the use of semi-rigid joints in steel frames for buildings. Both aspects are important to ensure the continued popularity of steelwork construction, which for buildings started over 100 years ago. The early use of multi-storey steel frame building construction started in 1884 when the 20-storey Home Insurance Building in Chicago was built with a steel frame encased by masonry[1.4]. Today the main advantages of steel systems are seen as follows[1.5]:

- The tensile and compressive strength and stiffness of steel produce a system with less massive construction than its rivals and is therefore more efficient in use of space.

- Steel construction easily provides large open spaces with a frame that can span up to 20 m.
- For the floor slabs, the need for temporary formwork is eliminated with the use of profiled metal decking which also provides space for electrical supplies in the building.
- Steelwork increases the speed, accuracy, and quality of the construction as fabrication can be done in the shop.
- Steel elements may be readily combined with concrete to provide composite action in which each material is used in its most appropriate manner.
- The natural form of steel can be used to achieve architectural satisfaction.

The most common reason though for selecting steel for a multi-storey frame is argueably to cut the construction time on site. The designer will seek to shorten time by specifying standard bolt sizes and by the use of straight-forward standard connections and by avoiding too many changes in section sizes. Within these constraints however, the designers should still seek the optimal method of design whilst ensuring the required levels of safety. Economic comparisons have shown that office buildings are cheaper as steel-framed structures than in concrete in overall construction[1.6]. A study by British Steel has concluded the following[1.7]:

- Steel frames provide the fastest form of construction.
- Composite beams and slabs are the most cost effective form of floor construction for basic building structures.

- A steel slimflor is more cost effective compared with reinforced concrete if considering total building cost.
- For long spans which provide column-free spaces a steel structure has an advantage of about 2 to 3% in terms of the total building costs.

Another advantage of steel over concrete is that steel buildings are more easily altered, extended, adjusted, and repaired for future needs[1.8].

The manner in which lateral stability is provided influences the cost of the framework. Frames are traditionally classified into two types; namely braced and unbraced, based on their means of providing this stability. The quantitative classification as braced or unbraced depends on the relative stiffness of any bracing system provided to limit horizontal deflection. A bracing system can be obtained by triangular trusses, concrete cores, lift shafts, shear walls, or by a very stiff region within the overall frame.

1.2 Braced frame.

For a frame to be classified as braced, the bracing system provided should be at least five times stiffer than the stiffness of the frame itself[1.9]. In Eurocode 3 (EC3) the frame is classified as braced when the bracing system reduces the horizontal displacement by at least 80%[1.3]. To meet this requirement the stiffnesses of the two systems (unbraced frames and braced frames) have to be compared and the following relationship has to be satisfied: $K_b \geq 5K_a$ where K_a and K_b are lateral stiffness spring constant for unbraced and braced frame respectively. In checking for ultimate limit state, it is important to make sure that the bracing system is capable of transferring the factored loads down to the foundations.

1.2.1 Design approach for braced frames.

Conventional design methods for braced steel frames for buildings can be categorised as either "simple" or "rigid", depending on the assumed behaviour of the joints[1.10]. Until recently, this categorisation was based on engineering judgement. A braced frame with pinned joints is assumed to contribute no end moment to the column, except in U.K practice for an "eccentricity" moment resulting from the beam reaction (Fig. 1.2); the effects of this moment are to some extent offset by the use of a column buckling length less than the true length, as described in BS 5950: Part 1[1.10]. As a result, the beam is designed as a heavy or deep section; in contrast, a column, designed for an axial load and "the eccentricity" moment, has (except in structures of many storeys) a light section. In designing braced frames, nominally-pinned joints (Fig. 1.3) are most commonly used because:

- they are easy to fabricate and save time; they cut the construction cost
- manuals on standardised nominally-pinned joints[1.11] are available to ease the design.

Another method of designing a braced frame is to use rigid joints[1.12]. This method may be used to design beams with a lighter or less deep section. The use of rigid joints in a braced frame contributes a significant amount of moment to the column which results in heavy column sections. Rigid joints are difficult and time consuming to fabricate, and usually require substantial stiffening to the column web to resist the large forces arising from the beam end moments; stiffening may also be judged necessary to realise the assumption of rigid behaviour. Although welded joints (see Fig. 1.4) can be categorised as rigid[1.12], for large welds higher residual stresses and greater distortion occur which increase the tendency of crack formation on weld metal and base metal[1.13]. Overall, rigid joints are not economical and not commonly used in multi-storey construction.

1.3 Unbraced frame.

A steel frame which does not satisfied the criterion for a braced frame is classified as unbraced. Although an unbraced frame may be treated as a three-dimensional entity, it is usually idealised as a series of two-dimensional frames that resist loading (horizontal and vertical) in each plane primarily by bending action. In practice it is often arranged though that the frames are braced against horizontal displacements in one direction to simplify the behaviour and to avoid as far as possible bending action about the minor axes of the column sections. Unbraced frames may also be "sway" frames in which second-order effects need to be accounted for. The "P-Δ" effect (Fig. 1.5) changes the distribution of internal moments and forces and results in a lowering of the load level at collapse. In unbraced frames, it is important to note that limitations of sway under service loading need to be satisfied, as well as the ultimate strength. These concern both the interstorey drifts and the structure as a whole. For example, the limits recommended by Eurocode 3[1.3] are $h/300$ for the interstorey drifts but $h_o/500$ for the structure as a whole, where h is the storey height and h_o is the overall height of the building.

1.3.1 Design approach for unbraced frames.

As loads in unbraced frames are to be resisted by bending action of the frame's members without the need of a bracing system, the most common design approach is to use rigid joints. For unbraced frames, the main design consideration is to limit sway, to avoid unacceptable deflections under service load and to avoid premature collapse by frame instability[1.14]. This can be done by using stiff joints and appropriate member sections. Fully welded connections are the closest approach to a truly-rigid joint but result in expensive fabrication costs. An extended end plate, welded to the beam and bolted to the column, provides a more reasonable form, but both welded and bolted joints are likely to require stiffening in the tension and compression zones of the column webs, and possibly in shear (see Figure 1.6). This may be to increase moment capacity or to reduced the sway, but this increases the fabrication cost even more. Generally, unbraced frames designed with rigid joints are not commonly adopted unless to meet an architectural requirement that no bracing system be allowed.

1.4 Sway and non-sway.

Second order effects increase the complexity of design. This is because in "second-order analysis", the equilibrium and kinematic relationships are dependent on the deformation of the structure[1.15]. It is important to at least consider including second-order effects in the design of unbraced frames to ensure the stability of the structure.

A frame is classified as non-sway, even if unbraced, provided that its stiffness is sufficient enough for second-order effects to be neglected[1.16]. For a rigid jointed frame, the deflection δ of every individual storey of height h, with notional horizontal loading as stated in clause 5.1.2.3 of BS 5950, Part 1[1.10], should satisfy the following criteria[1.15],

- Clad frames where cladding is not considered as a stiffener, $\delta \leq \frac{h}{2000}$.
- Unclad frames or clad frames where cladding is considered as a stiffening effect, $\delta \leq \frac{h}{4000}$.

These criteria are to meet the requirement that the elastic critical load λ_{cr} should be greater than 10 for a clad frame and greater than 20 for an unclad frame[1.15]. For clad frames, it follows that rigid-plastic design may be used provided λ_{cr} is at least ten times the rigid-plastic collapse load factor λ_p.

The similar definition of a no-sway frame provided by Eurocode 3 states: "A frame can be classified as non-sway if its response to in-plane horizontal forces is sufficiently stiff for it to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of its nodes"[1.3]. It follows that a rigid jointed multi-storey frame which does not satisfy the above criteria should be classified as a sway frame, even if the frame is braced. The classification as sway or non-sway can be summarised as shown in Figure 1.7[1.2], in which no deflections are shown to represent the non-sway case. The consequences of the classification for design are now considered further.

1.4.1 Design approach for non-sway frames.

For non-sway frames, the only design requirements related to stability that need to be checked are the resistances of individual members to flexural buckling, lateral-torsional buckling and local buckling. Design codes such as Eurocode 3[1.3] and BS 5950[1.10] provide rules to avoid premature collapse by such modes. In some frames, plastic hinges form below the required ultimate limit state load and their influence on stability in the member needs to be considered.

1.4.2 Design approach for sway frames.

By definition, significant second-order effects arise from the behaviour of the frame as a whole, and therefore design methods must address this aspect of behaviour, as well as the forms of instability which affect non-sway frames. When a sway structure becomes unstable, it is often related to the overall deformation of the structure. In a second-order analysis, a non-linear load-deflection curve can be determined by taking account of the change in the behaviour of the structure. The analysis usually proceeds in a step-by-step incremental manner with repeated formulation of the equilibrium and kinematic relationships.

In general, nonlinearities in structures can be divided into two forms, namely, geometrical nonlinearity and material nonlinearity. In term of multi-storey framed structures, geometrical nonlinearity may be restricted to in-plane P-Δ effects. The geometrical non-linear effect can be considered in the analysis by stability functions in formulation of the frame's stiffness[1.16] or by the use of a geometrical stiffness matrix in a finite element formulation[1.16]. Material non-linearity commonly results from the formulation of plasticity.

Figure 1.8 shows a comparison of the load-deflection behaviour of a rigid-jointed plane frame based on different assumptions in analysis:

- First-order elastic analysis - neglects the effects of both the changes of geometry and the yielding of the material.
- Second-order elastic analysis - considers the effects of changes of geometry and instability.
- Plastic-mechanism load - obtained by a simple plastic analysis such as virtual work, assuming discrete plastic hinges.
- First-order, elastic-plastic hinge analysis - neglect the effects of the change of geometry.
- Second-order, elastic-plastic hinge analysis - consider the effects of the change of geometry in plastic analysis by taking into account the increase in sway deflection (the P-Δ effect).
- Second-order, spread-of-plasticity analysis (Plastic-Zone Theory) - residual stresses, initial imperfections and strain hardening are all taken into consideration.

At low levels of load these various approaches result in approximately the same behaviour, with the differences becoming in some cases very noticeable at higher load levels (see Figure 1.8).

Design codes permit plastic design of sway frames provided that frame instability effects are considered by carrying out an elastic-plastic sway analysis. The analysis should allow for the formation of plasticity (usually idealised as plastic hinges), the moment-rotation response of any semi-rigid joints, and second-order effects often accounted for by use of stability functions[1.15].

In continuous frames, the interaction of elastic buckling characteristics and reduction in stiffness due to the development of plasticity may be used to determine more approximately the resistance of the frame[1.10][1.14]. The Merchant-Rankine approach[1.17] is based on such interaction. As modified by Wood, it is included in BS 5950[1.10]. If the design load level is taken as unity, then for a clad frame: $\lambda_p \geq \frac{0.9 \times \lambda_{cr}}{\lambda_{cr} - 1}$.

1.5 Semi-rigid or partial strength connections.

As already mentioned, steel frames for buildings have usually been designed on the basis that beam-to-column joints are either pinned or rigid. The actual stiffness though will often fall between these extremes, giving what is generally termed 'semi-rigid' behaviour. A joint may also have a moment resistance less than that of the connected beam; such behaviour is termed 'partial-strength' by Eurocode 3[1.3]. Such joints offer the designers the possibility of including the moment resistance of beam-to-column connections in an equivalent plastic hinge analysis of the frame[1.3] as well as performing elastic design based on the joints' stiffness. Frames which contain semi-rigid or partial-strength joints are termed 'semi-continuous' by Eurocode 3[1.3]. This code has encouraged the use of this approach to design by including a method to predict both the rotational stiffness and moment resistance of some types of joint. Some national codes, for example BS 5950 (1990)[1.10], also permit semi-continuous design but the British Standard fails to provide a method to predict the joints' properties.

In semi-continuous framing, the nature of joints which are not rigid may reduce further the stiffness of the frame. The increase in deflections in an unbraced frame results in second-order effects having greater significance at the ultimate limit state. The extent to which the stiffness is lost and the deflection is increased depends on the stiffness of the connected members, types of joints, and orientation of the column axis. Beam-to-column connections generally have non-linear moment-rotation curves. Initially, the connections have a stiff initial response which is then followed by a second phase of much reduced stiffness. This second phase is due to in-elastic deformation of the connections' components or those of members of the frame in the immediate vicinity of the joint. These deformations need to be accounted for because they contribute substantially to the frame displacements and may affect significantly the internal force distribution. The structural analysis needs to account for this non-linearity of joint response to predict accurately both stiffness and resistance for a semi-continuous frame in effect the joint behaviour is a form of material non-linearity.

Studies of the non-linear behaviour of connections have been summarised by Nethercot[1.18]. Four general techniques can be used to represent the non-linear behaviour of connections:

1. Mathematical expressions.
2. Simplified analytical models.
3. Mechanical models.
4. Finite element analysis.

1.5.1 Mathematical expressions.

These predict connection behaviour by empirical formulae within the range of data specified. The results of this approach accurately represent the M-Φ curve of the connections within this range. However, these methods cannot be used for connections outside the specified data and may not show an exact overall joint response.

A typical empirical formula for non linear behaviour can be represented by a standardised polynomial expression as follows: $\Phi = c_1(kM) + c_2(kM)^3 + c_3(kM)^5$ where the values of the constants "c" are obtained by using curve fitting, such as the least-squares method. The parameter "k" is a scaling factor which depends on relevent geometrical parameters for the particular joint configuration, obtained by considering a family of experimental M-Φ curves. Approaches to evaluate the parameters "k" have been developed by many researchers[1.19][1.20][1.21]; Sommer's method[1.19] will be described in more detail later in this Chapter.

Empirical formulae for non-linear behaviour of connections can also be expressed as an exponential function, which has the advantage of ensuring positive slope. This response corresponds to the observed rotational stiffness of connections. The exponential function adopted by Ramberg-Osgood[1.22] and modified by Ang and Morris[1.23] is as follow:

$\frac{\phi}{\phi_o} = \frac{kM}{[kM]_o}\left[1+\left(\frac{kM}{[kM]_o}\right)^{n-1}\right]$ where the ϕ_o and $[kM]_o$ are defined in Fig. 1.9 and the exponent "n" determines the shape of the curve.

Another technique to avoid the possibility of developing negative slope is the "B-spline", suggested by Jones[1.24]. The resulting formula is as follows:

$$\phi = \sum_{j=0}^{3} a_j M_j + \sum_{j=1}^{m} b_j \left(\left\langle M - M_j \right\rangle\right)^3$$

where m is the number of junctions of the multi linear curve and

$\left\langle M - M_j \right\rangle = M - M_j$ for $(M - M_j) > 0$,

$\left\langle M - M_j \right\rangle = 0$ for $(M - M_j) < 0$.

1.5.2 Simplified analytical models.

These models use the basic concepts of structural analysis such as equilibrium, compatibility, and properties of materials to give simplified models of the main elements in various types of beam-to-column connections. This can be achieved as follows:

1. Identify from test results the significant aspects which contribute to deformation in the connection.

2. Predict the initial stiffness of the connections from elastic analysis.
3. Predict the ultimate moment capacity from plastic mechanisms developed in the main element(s).
4. Verify the equation developed by comparison with test results.
5. Formation by curve fitting of the complete M-Φ curve from the predicted initial stiffness and ultimate moment capacity.

This simplified model had been adopted by Chen and Kishi[1.25] to study the behaviour of web cleats(single or double sided), flange cleats and combined web and flange cleat connections. The results for initial stiffness and the ultimate moment capacity from each of the connections are applied in a power expression of the form $\phi = cM^{\alpha}$, known as the Richard formula[1.26].

The method was further extended to the study on the behaviour of flush end plate connections by Johnson and Law[1.27] with proposals on the prediction of the inital stiffness and plastic moment capacity of the connections. The so-called "component method" used as the basis of Eurocode 3's rules for steel joints [1.3] can also be regarded as a simplified analytical model. Because of the importance of this method, it will be described separately in Section 1.5.5.

1.5.3 Prediction by mechanical models.

The objective of this approach is to directly predict the joints' response as an M-Φ curve, obtained without the need to depend on assumed patterns of behaviour. The models are considered to be adequate and reliable to the study of steel connections. This can be achieved by considering the connection/joint acts as a set of rigid or deformable components. However, the accuracy of this method depends on the assumptions taken for load-deformation laws of each of the connection components. Therefore, a full understanding is needed for the behaviour of each of the components which can be achieved from experimental and numerical research.

This approach was adopted by Kennedy and Hafiz[1.28] to study the behaviour of header plate connections. A similar method was used by Wales and Rossow[1.29] to model the behaviour of double web cleat connections. The method was further extended by Richard[1.30] to predict the behaviour of all types of cleated connections with bending and shear. The method was also adopted for fully welded connections by Tschemmernegg[1.31].

1.5.4 Finite element analysis.

This analysis is becoming more popular in recent studies for numerical modelling of nonlinear behaviour of connections. This is due to better computer facilities capable of speedy analysis of complex problems. Recent development of software such as ABAQUS[1.32] which is capable of solving problems by a standard nonlinear finite element analysis encourages the use of this approach. The advantage of this analysis is that it does not rely on an understanding of the detailed behaviour of the connections which depend on test results. Another advantage is that it is capable of including both geometric and material non-linearity with the choice of selecting the interface elements and constraint conditions in the analysis. This method needs each component of the connection/joint such as reinforcement, the shear studs (including slip between the slab and the beam), bolts (including slip), the steel beam and column (including buckling and plasticity) to be properly represented. This method has been proved successful to model different types of bare steel connections[1.33][1.34]. With its advantages it is likely to be preferred for modelling of composite connections which have many variables. Ahmed[1.35] has adopted this method in modelling composite connections and demonstrated accurate results by comparing with test results and simplified methods.

1.5.5 Prediction of connection response: Annex J of Eurocode 3.

To permit design of semi-continuous framing as an everyday practice, it is essential to be able to assess the stiffness and the moment resistance of a joint. Such a method is given in Eurocode 3[1.3]. Annex J of this document provides formulae to calculate the resistance and the stiffness of beam-to-column joints, framing into the column flange. Eurocode 3 describes the relationship between the bending moment and joint rotation in the following manner, shown in Fig. 1.10. The initial rotational stiffness $S_{j,ini}$ for the elastic range is taken to apply up to a level of 2/3 of the design moment resistance $M_{j,Rd}$. Beyond the elastic range, the stiffness diminishes until attaining a plastic moment resistance $M_{j,Rd}$. Once the moment in the joint reaches this value a constant plateau is assumed.

The original rules in Eurocode 3[1.3] were revised shortly after publication, so that the secant stiffness to be used in global analysis is taken as a fraction of the initial stiffness, rather than vice-versa. Revised rules are now available[1.36]. Now for global analysis the rotational stiffness S_j of a joint is simpified as half of initial rotational stiffness, $S_{j,ini}/2$[1.36].

The rotational stiffness of a joint may be determined from the flexibility of each of its basic components, the so-called "component method". It considers the joint not as a whole but as a set of individual components each having its own strength and stiffness. By assembling the components, the rotational stiffness of the complete joint can be calculated using the formulae summarised below, with particular reference to bolted connections.

The effective stiffness coefficient $k_{eff,r}$ for a bolt row r should be determined from:

$$k_{eff,r} = \frac{1}{\sum_i \frac{1}{k_{ir}}}$$

where k_{ir} represent the stiffnesses of the basic components, such as the end-plate in bending and the bolts in tension which influence the stiffness at that level. They are modelled as springs[1.37] in series and parallel (Fig. 1.11(a)). Figure 1.11(b) shows the deformations per bolt row of components represented as 3,4,5,and 7 added together and converted into an effective stiffness coefficient $k_{eff,r}$ where r is the index of the row number.

For end-plate connections with more than one bolt row in tension, all of these effective springs per bolt row are represented by a single equivalent stiffness coefficient k_{eq} (Fig 1.11(c)) determined from:

$k_{eq} = \frac{\sum_r k_{eff,r} h_r}{z}$ where; h_r is the distance between bolt row r and the centre of compression and z is the lever arm: $z = \frac{\sum_r k_{eff,r}.h_r^2}{\sum_r k_{eff,r}.h_r}$.

For the overall stiffness, account should also be taken of the flexibility in the compression zone and possibly shear. The resulting formula given in the revised Annex J is:

$S_j = \frac{E \times z^2}{\mu \sum_i \frac{1}{k_i}}$ where:

k_i is the stiffness coefficient representing each significant zone of deformation in the connection,

μ is the stiffness ratio $S_{j,ini} / S_j$,

$S_{j,ini}$ is the value of S_j when the moment $M_{j,Sd}$ is zero.

The stiffness ratio μ should be determined from:

$\mu = \left[\frac{1.5M_{j,Sd}}{M_{j,Rd}}\right]^{\psi}$ but $\mu \geq 1$

in which the coefficient ψ is obtained from the codes; the values have been determined from test results.

The present author will address the response of minor-axis beam-to-column joints, for reasons explained in Section 1.8. For the analysis of such connections, the following components from Eurocode 3 are relevant:

(i) Stiffness coefficients for end-plate, single bolt-row in tension: $k_5 = \frac{0.85 l_{eff} t_p^3}{m^3}$

where

t_p is the thickness of the end plate,

l_{eff} is the length of a Tee-stub equivalent to the real pattern of yield lines which occur around the bolt positions in the end plate.

For the first bolt-row below the tension flange of a beam (to be utilised in Chapter 4), the effective length is taken as the smallest of $2\pi m$ for circular yield-line patterns (Fig.1.12(a)) and αm for non-circular yield-line patterns(Fig. 1.12(b)). The value of α reflects the influence of the flange and web of the beam on the shape of the yield line pattern[1.38]:

$\lambda_1 = \frac{m}{m+e}$ and $\lambda_2 = \frac{m_2}{m+e}$ where m, m_2 and e are shown in Fig. 1.12(b).

(ii) Stiffness coefficients for bolts, single bolt row in tension: $k_7 = \frac{1.6A_s}{L_b}$

where

A_s is the tensile stress area of the bolt,

L_b is the elongation length of the bolt, which may be taken as equal to the grip length (total thickness of material and washers), plus half the sum of the height of the bolt head and the height of the nut.

1.6 Scope of present work on braced frame with partial strength connection

By providing detailed rules to calculate joint properties, Eurocode 3 encourages the use of semi-continuous construction. Studies on braced frames are known to result in significant savings in frame weight[1.39],[1.40] if this form of construction is adopted. To be attractive to designers though, calculation methods need to be straightforward and savings are required in overall frame cost, not just weight. The study in Chapter 2 describes the advantages of plastic design using partial-strength joints. This approach has been used in comparison with simple design. The effect on the design moment of beam of using semi-rigid connections is shown in Fig. 1.13[1.39]. It can be seen that by introducing semi-rigid connections, the maximum elastic moment at the centre will be reduced compared with the simple case. The reduction of moment is less though than that for a fixed ended beam, and a balanced distribution of moments can be achieved by the use of an appropriate connection stiffness, S_j relative to that of the beam. The variation of moment with the relative stiffness is as shown in Fig. 1.14[1.39]. To minimize the beam's design moment may however sacrifice overall economy of construction by increasing the column size due to greater moments in these members and by greater fabriction cost.

1.7 Wind-moment design for unbraced frames.

1.7.1 Introduction.

For the design of unbraced sway frames at ULS, it is possible to use advanced methods based on interaction of elastic buckling characteristics and the reduction in stiffness due to plasticity[1.3][1.10]. The joints are assumed to be rigid and full-strength. However, other less-sophisticated methods apparently based on purely elastic behaviour are also available. One approach, termed the "wind-moment" or "wind-connection" method[1.41], is often used in the U.K. The method is known as "Type 2 Construction" in the U.S.A[1.42]. In its simplest form the "wind-moment" method assumes[1.41]:

1. under gravity load, the connections act as pins (Fig. 1.15(a)); this means that the beam members are designed as simply supported with no moments transferred to the column, other than nominal "eccentricity" moments;.
2. under wind load, the connections behave as rigid joints, with points of contraflexure at the mid-height of columns and mid-length of beams (Fig. 1.15(b)).

Members are proportional initially to resist gravity load. The internal forces and moments due to gravity load and wind (Fig 1.16(a) and Fig. 1.16(b)) are then combined in appropriate load cases. The design at the ultimate limit state is completed by amending the initial section sizes and other details for the members and connections, to withstand combined load effects.

The advantage of the method is its simplicity. The frame is considered as statically determinate with internal moments and forces not dependent on the relative stiffness of the members. The need to repeat the analysis to correspond to changed section sizes is thereby avoided. The beam sections generally have the same size for all the floors since the mid-span moment due to gravity load usually controls the design, thus simplifying the construction of the building. In contrast, for fully continuous construction with rigid joints, the beam sections tend to be different at the various floor levels[1.43]. A further advantage is that connections usually do not require the web stiffening often associated with rigid joints; this is because they are designed for moments due to wind loading only. As a result, fabrication costs are reduced and designers have greater freedom in the positioning and size of beams which frame into the column web.

For serviceability, sway deflections are calculated assuming connections are rigid. Second-order analysis due to the "P-Δ" effect is not included in the calculation. It is assumed that these can be accounted for by using effective column lengths greater than the true lengths, for axes about which sway can occur.

The method described above has been used extensively, and design rules consistent with BS 5950: Part 1: 1990[1.10] have been published[1.41]. These were developed in conjunction with an analytical study of typical frames designed by the method[1.44]. Despite its widespread use though, the method cannot be fully accepted as a generally applicable approach. The scope of the rules was therefore restricted to that of the study. In particular they apply to steelwork which can be idealised as a series of unbraced plane frames which are effectively braced against out-of-plane sway at roof level and each floor level as shown in Figure 1.17. Within each plane frame the column sections should be oriented such that loads in the plane of the frame tend to cause bending about the major axes. This represents an unwelcome restriction on the forms of structure to which the "wind-moment" method can be applied. Studies were therefore required to verify the method when the structure can sway about both column axes.

1.7.2 Partial strength joints.

In wind-moment design, the connections are proportioned to resist the moments generated by horizontal forces. As the beam section is usually governed though by mid-span gravity moment, the connections are designed to a lower moment than the beam sections (Fig.1.18) and are therefore "partial strength" within the context of Eurocode 3 Part 1.1[1.3]. Because in reality the elastic moments applied to the connections are considered higher than the wind-moments, a prerequisite of the connections is that they should be "ductile" because plastic hinges can be expected to form in them below the design load level for ULS. This has led to the development of a range of standard ductile connections for "wind-moment" frame[1.38][1.45].

A survey by Jenkins[1.46] showed that the end plate connections were widely used, to an extent which justified standardisation in design and detailing. Recent research [1.38][1.45][1.47] on partial strength connections has led to a new standardised arrangements which provide assured ductility and moment resistance from the connection.

The standard tables[1.38] provide immediate information for partial strength connections in wind moment design. The standard connections applicable to wind-moment frames are beam-to-column connections made with extended or flush end plates (Fig 1.19(a) and Fig 1.19(b)). In order to achieve a ductile response, the connections employ relatively thin end plates (12-15mm thick). As a result of these and as no stiffening is provided, these joints are relatively flexible. With this behaviour, it is still necessary to meet the limitations on sway at SLS and avoid the premature collapse of frames due to instability. Therefore, a study was carried out by Brown[1.47] on the performance of major-axis wind-moment frames with ductile connections. The results on Brown's studies showed that wind-moment frames performed satisfactorily when designed with the standard connections and were capable of withstanding the applied loads without any stability problems[1.47].

1.7.3 Scope of present work on wind-moment design for frames unbraced about both column axes.

Framing unbraced about the minor axis of the column is inherently less stiff than major axis framing, because of the common use of H-section columns with their smaller flexural rigidity for bending in the plane of the flanges. However, the need to position stiff cores or to arrange bracing limits architectural freedom, and there is therefore demand for frames to be unbraced about both axes. Wind action will then result in sway about both major and minor axes of the column sections, and torsion. Although the existing rules for "wind-moment" design have been shown to result in frames of adequate resistance to major-axis sway[1.47], if sway occurs about the minor axis, the following additional concerns arise:

1. The form of minor-axis connection must provide reasonable moment resistance and stiffness.
2. The frame must be stable against minor-axis sway, despite the low flexural rigidity exhibited even by Universal Columns bent in this way.

3. In frames supporting precast units, the minor axis beams may remain as little more than tie members even when designed for wind moments, with consequent absence of appropriate stiffness to ensure frame stability. To ensure these beams are of adequate section, check on lateral torsional buckling is needed in accordance with BS 5950[1.10], the formulae are stated in Section 1.7.3.1.
4. The frame must be checked for adequate member stability now with moments applied about the weak axis. For design to BS 5950[1.10], the formulae are stated in Section 1.7.3.1.

In Chapter 3 the author reports studies on the wind-moment method for design of frames unbraced about both axes, adopting a form of connection with the beam connected to a thick end plate welded to column flanges as shown in Figure 1.20. This very stiff end plate is considered not deformable, but ductility and flexibility are provided by the end plate to the beam. The initial stiffness used in this connection is determined by adapting from a method proposed by Brown[1.47]. For minor axis connections used in Chapter 3, the equation was modified with the term corresponding to the column flange being omitted, as described in Section 1.7.3.2. For the design of frames bending about the minor axis, Anderson and Islam's method[1.48] was adopted to limit sway deflections; the method is summarised in Section 1.7.3.3. The frames were further checked with software written by Kavianpour[1.49] for first and second order analysis by taking into account the non-linear behaviour of the connections. The study of the behaviour of unbraced steel frames with semi-rigid connections[1.49] was extended from an early study done by Anderson. Proposed rules to improve the stiffness of the connections are presented by the author in Chapter 3. For convenience in later presentation of the author's work, the member stability checks of BS5950 and the works of Brown, Islam and Kavianpour are now briefly described. Further work by the present author on wind-moment design with composite beams is justified in 1.9.

1.7.3.1 Lateral torsional buckling formulae for beams and overall stability formulae for columns.

Formulae for lateral torsional buckling on beams.

Equations used for checking resistance to lateral torsional buckling are as follows[1.10]:

$$\overline{M} \leq M_b$$

where

$\overline{M}$ is the equivalent uniform moment,

M_b is the lateral torsional buckling resistance moment.

The equivalent uniform moment, $\overline{M}$ is given by:-

$$\overline{M} = mM_A$$

where

M_A is the maximum moment acting on the member,

m is an equivalent uniform moment factor.

The buckling resistance moment, M_b is given by:

$$M_b = S_x p_b$$

where

S_x is the plastic modulus of the section about the major axis,

p_b is the bending strength determined from Table 11 of BS 5950 Part 1[1.10].

The value of λ_{LT} need to be determined before calculating the value of p_b

$\lambda_{LT} = nuv\lambda$

where

n is a slenderness correction factor taken to be equal to 1,

u is a buckling parameter,

v is a slenderness factor,

λ is the minor axis slenderness: $= L_E / r_y$ where

L_E is the effective length taken to be 0.85 L, assuming partial end restraint,

r_y is the radius of gyration about the minor axis of the member.

Formulae for local capacity check on columns.

The formula given below is used

$$\frac{F}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

where

F is the applied axial load in member,

A_g is the gross cross-sectional area,

M_x is the applied moment about the major axis at critical region,

M_{cx} is the moment capacity about the major axis in the absence of axial load,

M_y is the applied moment about the minor axis at critical region,

M_{cy} is the moment capacity about the minor axis in the absence of axial load.

Formulae for overall buckling check on columns.

The formula given below is used

$$\frac{F}{A_g p_c}+\frac{m_x M_x}{M_b}+\frac{m_y M_y}{p_y Z_y}\leq 1$$

where

F is the applied axial load in member,

A_g is the gross cross-sectional area,

M_x is the applied moment about the major axis at critical region,

M_y is the applied moment about the minor axis at critical region,

p_c is the compressive strength,

m_x and m_y is the equivalent uniform moment factor obtained from Table 18 of BS 5950 which depend on curvature of the beam deformed,

M_b is the buckling resistance moment capacity (about major axis),

Z_y is the elastic section modulus about the minor axis,

p_y is the design strength.

1.7.3.2 Initial stiffness by Brown's method.

Brown's method[1.47] approached the prediction of stiffness by identifying the main components which contributed to flexibility of the connection. Two main components are:

- the end-plate
- the column flange.

The prediction equation is based primarily on the summation of the flexibilities of each component. The flexibility of other components, such as bolts, is recognised by the inclusion of a coefficient to enhance the overall flexibility. The result of the rotational stiffness of a steel connection is as follows:

$$K_j = \frac{1}{\frac{0.7}{0.135 l_a^2}\left(\frac{m_{cf}^2}{t_{cf}^3} + \frac{m_{ep2}^3}{m_{ep1} t_{ep}^3}\right)}. \tag{1.1}$$

The method was validated by comparisons against tests on end plate joints[1.47]. However, for the equation to be adopted for the minor axis connections considered by the author in Chapter 3, the equation is modified to

$$K_j = \frac{1}{\frac{0.7}{0.135 l_a^2}\left(\frac{m_{ep2}^3}{m_{ep1} t_{ep}^3}\right)}. \tag{1.2}$$

where,

l_a is the connection lever arm. For flush end plate joints with one bolt row the lever arm is the distance from the bolt row to the compressive force acting through the centre of bottom beam flange. For an extended end plate the lever arm distance is from the top beam flange to the centre of bottom beam flange,

$m_{ep1} = \frac{G}{2} - \frac{t_{wb}}{2} - 0.8L_w, \leq m_{ep2},$

$m_{ep2} = E_{1t} - t_{bf} - 0.8L_f,$

$t_{wb} =$ beam web thickness,

$E_{1t} =$ vertical distance between the centre-line of the top row of bolts and the outer most surface of the top flange of the beam,

L_w = leg length of the web weld,

L_f = leg length of the flange weld.

1.7.3.3 Anderson and Islam method.

This method is presented for the design of multi-storey steel frames to limiting values of horizontal sway deflection. The frame is divided into statically determinate sub-frames by assuming points of contraflexure. Allowance for steelwork costs are then used, together with slope-deflection analysis, to derive equations for optimum design. This method is suitable for hand calculation. The accuracy of the design equations was found to be good by comparison with linear elastic computer analysis.

1.7.3.3.1 Design equations for the top storey.

The subassemblage shown in Figure 1.21 was used to derive the design equations as stated below:

$$I_{1,2} = \frac{P_1 h_1 I_{2,2}}{P_1 h_1 + P_2 h_2} \tag{1.3}$$

$$I_{2,2} = \frac{(P_1 h_1 + P_2 h_2) h_1 L_2^2 I_{3,2}}{24 E \Delta B I_{3,2} - P_1 h_1^3 (L_1 + L_2)} \tag{1.4}$$

$$I_{3,2} = \frac{\left[P_1 h_1^3 (L_1 + L_2) + h_1 L_2 \sqrt{\dfrac{P_1 h_1 (L_1 + L_2)\left[2 W_1 P_1 h_1 + W_2 (P_1 h_1 + P_2 h_2)\right]}{W_3}} \right]}{24 E \Delta B} \tag{1.5}$$

$$I'_{3,2} = \frac{I_{3,2}}{2} \tag{1.6}$$

where

P_1 is the total horizontal shear in the top storey columns,

P_2 is that in the storey below,

L_1 and L_2 are the span of the beams,

h_1, and h_2 are the height of the columns,

Δ equal the allowable sway over the storey height h_2,

B is the total width of the frame,

$I_{1,2}$ is the inertia of the upper beams in the storey,

$I_{2,2}$ is the inertia of the lower beams in the storey,

$I_{3,2}$ is the inertia of the internal designed column in the storey,

$I_{3,2'}$ is the inertia of the external designed column in the storey.

1.7.3.3.2 Design equations for intermediate storeys.

The subassemblage shown in Figure 1.22 was used to derive the design equations stated below. It is assumed that the total horizontal shear is divided between the bays in proportion to the widths.

$$I_{1,2} = \frac{(P_1h_1 + P_2h_2)\,I_{2,2}}{P_2h_2 + P_3h_3} \tag{1.7}$$

$$I_{2,2} = \frac{(P_2h_2 + P_3h_3)h_2L_2^2 I_{3,2}}{24E\Delta B I_{3,2} - P_2h_2^3(L_1 + L_2)} \tag{1.8}$$

$$I_{3,2} = \frac{\left[P_2h_2^3\left(L_1 + L_2\right) + h_2L_2\sqrt{\dfrac{P_2h_2\left(L_1 + L_2\right)\left[W_1\left(P_1h_1 + P_2h_2\right) + W_2\left(P_2h_2 + P_3h_3\right)\right]}{W_3}}\right]}{24E\Delta B} \tag{1.9}$$

$$I'_{3,2} = \frac{I_{3,2}}{2} \tag{1.10}$$

where

P_1 and P_3 are the total horizontal shear in the columns of the storeys immediately above and below,

P_2 is the total horizontal shear in all the columns of the storey being designed,

L_1 and L_2 are the span of the beams,

h_1, h_2, and h_3 are the height of the columns,

Δ equal the allowable sway over the storey height h_2,

B is the total width of the frame,

$I_{1,2}$ is the inertia of the upper beams in the storey,

$I_{2,2}$ is the inertia of the lower beams in the storey,

$I_{3,2}$ is the inertia of the internal designed column in the storey,

$I_{3,2'}$ is the inertia of the external designed column in the storey.

1.7.3.3.3 Design equation for the bottom two storeys of a fixed base frame.

The subassemblage shown in Figure 1.23 was used to derive the design equations stated below. The fixity of the base attracts more moment than the upper column. As a result, the design may be governed by the permissible deflection Δ_2 of the upper storey. The effect of fixed base is more pronounced when $h_2 = h_3$, and the bottom storey column inertia ($I_{4,2}$) than has to be made equal to $I_{3,2}$) to avoid reverse taper.

$$I_{3,2} = \frac{\left(P_2 h_2 + L_2 Y\right) h_2^2 \left(L_1 + L_2\right)}{24 E \Delta_2 B} \tag{1.11}$$

where

$$Y = \sqrt{\frac{3P_2h_3\left[W_1\left(P_1h_1 + P_2h_2\right) + 2W_2\left(P_2h_2 + P_3h_3\right)\right]}{\left(3W_3h_3\left(L_1 + L_2\right)\left(h_2 + h_3\right) - W_2L_2^2\right)}} \tag{1.12}$$

$$I_{1,2} = \frac{\left(P_1h_1 + P_2h_2\right)h_2L_2^2I_{3,2}}{\left(24E\Delta_2BI_{3,2} - P_2h_2^3\left(L_1 + L_2\right)\right)} \tag{1.13}$$

$$I_{2,2} = \frac{\left(P_2h_2 + P_3h_3\right)h_2L_2^2I_{3,2}}{\left(24E\Delta_2BI_{3,2} - P_2h_2^3\left(L_1 + L_2\right)\right)} - \frac{L_2^2I_{3,2}}{\left(6h_3\left(L_1 + L_2\right)\right)} \tag{1.14}$$

$$W_1 = \frac{k_{1,1}L_1^3 + k_{1,2}L_2^3 .... + k_{1,m}L_m^3}{2L_2^2} \tag{1.15}$$

$$W_2 = \frac{k_{2,1}L_1^3 + k_{2,2}L_2^3 .... + k_{2,m}L_m^3}{2L_2^2} \tag{1.16}$$

$$W_3 = \frac{\left[\left(k_{3,1} + k_{3,2}\right)L_1 + \left(k_{3,2} + k_{3,3}\right)L_2 + ...\left(k_{3,m} + k_{3,m+1}\right)L_m\right]}{\left(L_1 + L_2\right)} \tag{1.17}$$

where

P_1 and P_3 are the total horizontal shear in the columns of the storeys immediately above and below,

P_2 is the total horizontal shear in all the columns of the storey being designed,

L_1 and L_2 are the spans of the beams,

W_1, W_2, and W_3 are the cost factors for a member of inertia $I_{i,j}$,

h_1,h_2, and h_3 are the heights of the columns,

Δ equals the allowable sway over the storey height h_2,

B is the total width of the frame,

E is Young's Modulus,

$I_{1,2}$ is the inertia of the upper beams in the storey,

$I_{2,2}$ is the inertia of the lower beams in the storey,

$I_{3,2}$ is the inertia of the internal designed column in the storey,

$I_{3,2'}$ is the inertia of the external designed column in the storey.

1.7.3.4 Method used by Kavianpour to analyse steel frames with semi-rigid connections.

The computer software developed by Kavianpour[1.49] was an extension to an elasto-plastic program for frames with rigid connections written by Majid and Anderson[1.50]. The program was based on the matrix displacement method. The software is capable of analysing any combination of pinned connections, fully-rigid joints and semi-rigid joints with known M-Φ behaviour. Frames are analysed up to the collapse load for the plane frame. The program allows discrete plastic hinges to form in members and second order effects to be accounted for.

The behaviour of semi-rigid connections determined from tests, mathematical expressions or analytical models[1.51] is included in the analysis with the use of successive estimation on the secant stiffness for each connection. The secant stiffness is adopted because it is has the ability to reduce errors and assist convergance as the collapse load is approach by avoiding any need to use very low values of tangent stiffness.

The program includes the effect of axial force on frame stiffness by using stability functions which modify the slope-deflection equations, assuming small-deflection theory. The load is increased proportionally. For each load increment, iteration is necessary to ensure that the internal forces and moments, the plastic resistance moments and the secant stiffnesses are consistent. The analysis compares internal moments with the plastic resistance moments of the members. If the comparison indicates, to an appropriate tolerance, the formation of a plastic hinge in a section, the stiffness equations need to be modified. Such a hinge may also form in a joint if the M-Φ curve has a plastic plateau. The frame is finally taken to have reached the collape load when the determinant of the stiffness matrix becomes negative. Further details of this method are explained elsewhere[1.49,1.52].

1.8 Properties of minor axis joints.

The study of unbraced frames bending about the minor axes of columns necessitates knowledge of the structural properties of minor axis joints. An obvious form of such joints is to connect beam end plates to one or both sides of the web of the column (Fig. 1.24) but both experimental and theoretical investigation of such joints has been limited to date. No design methods are in common use. The analysis and design of this type of connection are more difficult than that for flange connections for the following reasons[1.53]:

- the maximum strength of the connection assemblage under unbalanced moment may be limited by the formation of a yield line type of mechanism in the column web
- the possible development of local buckling of the column flanges and web due to high axial forces in the column
- possible fracture of material of the assemblage due to concentration of much higher stresses at the tip of the column flange

- the space restricted by the column flanges may hinder easy erection.

Early research on connections to the minor axis was reported by Graham et al[1.54] where four-way beam-to-column moment connections were tested with for static load on symmetric web connections. Full scale testing was reported by Rentschler, Chen and Driscoll in a study whereby a pair of steel plates, to represent a beam's flanges, are welded to column sections[1.53][1.55][1.56]. Tests involving the column web were also reported on three dimensional joints with the use of bolted connections comprising end plates, double web cleat, and flange cleats[1.57]. Thick angles or end plates, and high strength bolts, were used to ensure that failure of the connections arose in the column web; thus behaviour and moment capacity can be determined. The results showed that the column web yielded and the rotation of the joint was due almost entirely to the flexibility of the column web.

Another series of tests was also reported by Kim who examined flush end plate connections with different geometry configurations, all connected to the column web[1.58]. These tests were arranged to include the deformation of column web, in a situation similar to an external column in a frame as shown in Fig. 1.25. The test specimens are listed in Figure 1.26, from which it can be seen that the parameters were varied in systematic manner. All bolts were M16 Grade 8.8, and the steelwork was to Grade 43 (nominal yield strength 275 N/mm^2). A flush end plate connection with either four or six bolts was used in each test, providing two or three rows of bolts as shown in Table 1.1. Further details of the test procedure, instrumentation, and materials tests are available elsewhere[1.58]. Graphs of moment against rotation for deformation due to the column web, end plate, and whole connection were reported. The results showed that the end plate thickness and bolt arrangement have little influence on the connection behaviour; however the depth of the beam has significant influence. Further details are given in Chapter 4, in which the present author describes the use of these tests to develop design methods for such joints.

Tests on minor axis connections were also carried out by Gomes[1.59]. The connections used in the tests were flush end plates, double web cleats, and flange cleats and were connected to the column web. Gomes used his test results to prepare a method for prediction of the moment resistance M_R of beam-to-column web connections[1.59][1.60]. Gomes' model predicts flexural and punching shear modes for the web of the column due to the bolts in tension. It substitutes the plastic mechanism of the web by an equivalent rectangle of b x C dimensions as shown in Figure 1.27(b). Gomes studied five modes of failure, three being flexural modes and two in punching shear:

- Flexural modes:

1. Local mechanism.
2. Global mechanism.
3. Mechanism under each bolt head (only for bolted connections).

- Modes with punching shear:

4. Punching shear around each bolt head or around the compression or tension zone.
5. Combined flexural and punching shear mechanism (only for thick column webs).

The plastic failure load F_{pl} in either the tension zone or the compression zone of the joint can be calculated by the formulation of Gomes work presented in Table 1.2. In Chapter 4 of this thesis, Gomes' formulae are applied to the connections tested by Kim so as to compare the experimental and the theoretical results.

To predict the stiffness coefficient of the web panel of the column, the author has adopted Sommer's method[1.19]. Details of the predicting procedure were discussed in Chapter 4.

1.8.1 Standardisation of moment-rotation curves using Sommer's method.

Sommer represented the moment-rotation curve in the form of a power series relating the rotation, Φ, to a scaled moment KM. The standardisation procedure involves the representation of the moment-rotation curves for all connections of a given type by a single function of the general form:

$$\Phi = \sum C_i (KM)^i \tag{1.18}$$

In polynomial form the non-linear expression adopted was:

$$\Phi = C_1(KM) + C_2(KM)^3 + C_3(KM)^5 + \ldots.. \tag{1.19}$$

In this study described in Chapter 4, the concern is to predict the initial stiffness. Therefore, the equation for rotation is considered as linear. Therefore the single function is modified by the present author as

$$\Phi = C_1(KM) + C_2 \tag{1.20}$$

where

Φ = rotational deformation of connection in radians,

C = constant,

K = standardisation depends on the size parameters for the particular connection considered, and

M = moment applied to the connection.

The standardisation constant, K, for a connection depends on the connection size parameters and has the form:

$$K = \prod_{j=1}^{m} p_j^{a_j} \tag{1.21}$$

where

p_j = numerical value of jth size parameter,

a_j = a dimensionless exponent which indicates the effect of jth size parameter on the M-Φ relationship, and

m = total number of size parameters.

The exponent a_j was determined by considering a family of experimental moment-rotation curves for connections differing only in parameter p_j as shown in Figure 1.28(a). For example, two set of connections each with M-Φ curves having the same parameter such as column web thickness but differ in the others. However, in this study, moment-rotation curves for connections are assumed to be linear as shown in Figure 1.28(b). At a particular rotation, each pair of curves was assumed to have the form:

$$M_1 / M_2 = \left[p_{j2} / p_{j1} \right]^{a_j} \tag{1.22}$$

where p_{j1} and p_{j2} are the numerical values of parameter p_j for connection 1 and connection 2 related to curve 1 and curve 2 respectively. Equation (1.22) can then be rewritten and solved for a_j:

$$a_j = \frac{\log\left(M_1 / M_2\right)}{\log\left(p_{j2} / p_{j1}\right)} \tag{1.23}$$

The exponent's a_j were calculated using equation (1.23), for several different rotation values, for every available combination of M-Φ curves. The mean value was then adopted from all pairs of curves for connections which differed in the value of only one parameter.

1.9 Steel frames with composite beams.

Composite beams are defined as steel sections which act compositely with concrete or composite slabs by the use of shear connectors[1.61]. Composite action changes the behaviour of the individual elements, so that in a simply supported beam the steel section acts mainly in tension and the concrete slab in compression[1.62]. The design of framed structures for multi-storey buildings to include composite beams nowadays has become popular, due to the greater stiffness and higher load bearing capacity. Composite construction makes possible a section of less depth, leading to a reduced height of structure and consequently less cladding for buildings and lower embankments for bridges[1.63]. Most steel frames for the commercial building sector in the U.K are now designed using composite construction. Previous rules for wind-moment design[1.41] did not address use of composite beams. The present author has therefore examined this subject. The approach is that adopted previously[1.41], whereby a series of frames have been designed by the method and then analysed "exactly".

Although the design of simple or continuous composite beams in braced frames is well-known, practical guidance on design methods for semi-continuous or unbraced construction is very limited. For unbraced construction the influence of cracking of concrete in hogging moment regions is uncertain. Eurocode 4 [1.64], ignores the effect of the cracking of concrete in the hogging moment regions as long as the top-fibre tensile stress σ_{ct} does not to exceed $0.15f_{ck}$ where f_{ck} is a concrete strength. For a tensile stress σ_{ct} exceeding $0.15f_{ck}$, the stiffness should be reduced to the so-called 'cracked' value based on the steel section and reinforcement in the slab. For braced frames as an alternative, beams can be designed as continuous by elastic analysis with redistribution permitted to allow for cracking of concrete and yielding of steel in the negative moment regions[1.63]. Designers can therefore treat the continuous composite beam as a uniform member of uncracked section. The degree of redistribution is in practice very difficult to predict because tensile stress in the negative moment region is influenced by the casting procedure and the change in temperature and shringkage. The value given in design codes depends upon the classification of the steel cross-section at internal supports. With sections of good rotation capacity high degrees of redistribution are permitted because at ULS the behaviour will be similar to that of a beam designed by plastic theory and therefore uninfluenced by casting sequence, shrinkage or temperature effects.

For unbraced frames, degrees of redistribution are not established, and in the author's study use has been made of the alternative cracked section method. For braced frames, the cracked section for negative moments in a composite beam is assumed at a distance of 15% of the total length of the span on each side of an internal support. This assumptions makes possible global analysis without iteration. Studies have been reported for assumed lengths of cracked section of other than 15%[1.65]. It was found though that by assuming 15% as a cracked length, the accuracy of moment redistribution results are within 5% for any proportion of the span between 8% and 25% of cracked length. In the author's work on unbraced frames, iteration has been necessary.

The author's work reported in Chapter 5 has considered the analysis of unbraced frames with composite beams and standard connections[1.38] allowing from the variation from sagging to hogging moment along these members. For wind-moment frames there is additional concern that if composite beams are used, the standard ductile connections, which relate to the size of the steel section, will be inadequate to provide resistance to the wind-moments and inadequate to provide stability.

1.10 The use of shear connectors on composite steel-beam.

The main disadvantage of composite construction is the need to provide shear connectors for composite action to be effective. In designing composite steel-concrete beams, it is important to understand the behaviour of the connection between these components. This must be designed to resist separation between the steel and concrete elements and as far as reasonably possible to maintain the same curvature in bending when acting compositely. Therefore mechanical shear connectors are introduced to link these materials to resist both tensile forces normal to the steel-concrete interface, and shear forces parallel to the steel-concrete interface. Shear connectors are usually welded to the top flange of steel beams to transfer the longitudinal shear from the slab to the steel beam. A beam with sufficient connectors so that slip can be ignored is considered to have "full interaction". However, "partial interaction" commonly occurs when a slab is constructed with a permanent formwork of profiled sheeting; due to space limitations as a result of the nature of the profiled sheeting shape fewer shear connectors can be provided. This is acceptable provided there is still sufficent flexural strength and stiffness in the beam. The most economical design is to provide the minimum shear connectors needed.

The most popular shear connectors used are headed studs ranging in diameter from 13 to 25 mm, and in length from 65 to 100 mm[1.66]. The stud is installed on the steel member by an automatic electrical welding procedure. The shank of the welded stud resists the longitudinal shear load whereas the head resists the uplift force. Figure 1.29 shows a wide variety of mechanical shear connectors with different shape, size, and methods of bonding[1.67]. All of them however, have the same function, to bond together the steel and concrete elements. The economy of composite beams however is dependent on the fabrication costs of the shear connectors. Studs with diameter exceeding 19 mm become significantly more expensive and difficult to weld[1.66]. Therefore a new shear connector for composite steel-concrete beams has been introduced to the market which can be installed without the use of welding. This connector is in a form of base plate welded to the end of the stud connector, and is installed on the steel member by a driven pin. Pinned shear connectors were tested by Matus and Jullien but with different geometrical characteristics[1.68] to those subsequently tested by the present author. The main advantage of using this connector is to speed up the installation procedure and to reduce the cost of composite construction. However, laboratory testing is needed to prove the efficiency and the strength according to the test procedure described in Eurocode 4[1.64] before acceptance can be achieved.

Push-tests are usually used to determine the behaviour of shear connectors because they are inexpensive and easy to carry out. The behaviour of shear connectors can be studied from the relationship between the shear force transmitted and the slip at the interface. However, caution need be taken when interpreting the results as the tests are not representing the actual situation in composite beam, which may lead to different strengths and modes of failure. A series of push tests designed to investigate the new shear connector has been performed and is described in Chapter 6.

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Test Number	Beam Section	Column Section	End plate Thickness	Number of Bolt
1	152x89RSJ	152x152xUC23	3	Initial test
2 3 4	152x89RSJ	152x152xUC23	6 8 10	4 bolts
5 6 7	254x102xUB22	152x152xUC23	6 8 10	4 bolts
8 9 10	254x102xUB22	152x152xUC23	6 8 10	6 bolts
11 12 13	305x102xUB25	152x152xUC23	6 8 10	6 bolts
14 15 16	305x127xUB42	203x203xUC46	6 8 10	6 bolts
17 18 19	254x102xUB22	152x152xUC37	6 8 10	6 bolts
20 21 22	254x102xUB22	203x203xUC46	6 8 10	6 bolts

Table 1.1 Test specimens

Table 1.2 Determination of the failure load F_{pl} in Gomes formulae.

Validity range: $\left(b/L\right) < 0.8$ and $0.7 \le \dfrac{h}{(L-b)} \le 10$

Constants:

$$m_{pl} = \frac{1}{4} t_w^2 f_y$$

$$\alpha = \frac{4}{1-b/L}\left(\pi\sqrt{1-b/L} + 2c/L\right)$$

$$k = \begin{cases} 1 & \text{if } (b+c)/L \ge 0.5 \\ 0.7 + 0.6(b+c)/L & \text{if } (b+c)/L \le 0.5 \end{cases}$$

LOCAL FAILURE: $F_{local} = m_{pl}\alpha k$

GLOBAL FAILURE:

$$F_{global} = \begin{cases} m_{pl}\left(\frac{2b}{h} + \frac{\alpha k}{2} + \pi + \frac{2h}{L-b}\right) & if\ \dfrac{h}{L-b} \ge 1 \\ m_{pl}\left(\frac{2b}{h} + \frac{\alpha k}{2} + \pi + 2\right) & if\ \dfrac{h}{L-b} \le 1 \end{cases}$$

PUNCHING SHEAR FAILURE:

case 1 : punching shear around n bolt heads: $F_{Q1} = n\pi d_m t_w \frac{f_y}{\sqrt{3}}$

case 2 : punching shear around the rectangular area: $F_{Q1} = 2(b+c)t_w \frac{f_y}{\sqrt{3}}$

COMBINED FLEXURAL AND PUNCHING SHEAR FAILURE:

$$F_{Q2} = \begin{cases} F_{local} & \text{if } t_w \le L/20 \\ 4m_{pl}\left[\frac{\pi\sqrt{L(a+x)}+c}{a+x} + \frac{2cx+x^2}{\sqrt{3}t_w(a+x)}\right] & \text{if } t_w > L/20 \end{cases}$$

where: a = L - b

and $x \ge 0$ determined by the iterative procedure:

$$x_{i+1} = -a + \sqrt{a^2 - 2ac + \frac{\sqrt{3}t_w}{2}\left[\pi\sqrt{L(a+x_i)} + 2c\right]}$$

if $x_{i+1} < 0$ take: $F_{Q2} = F_{local}$

PLASTIC FAILURE LOAD:

$$F_{pl} = \min\left(F_{locall}, F_{global}, F_{Q1}, F_{Q2}\right)$$

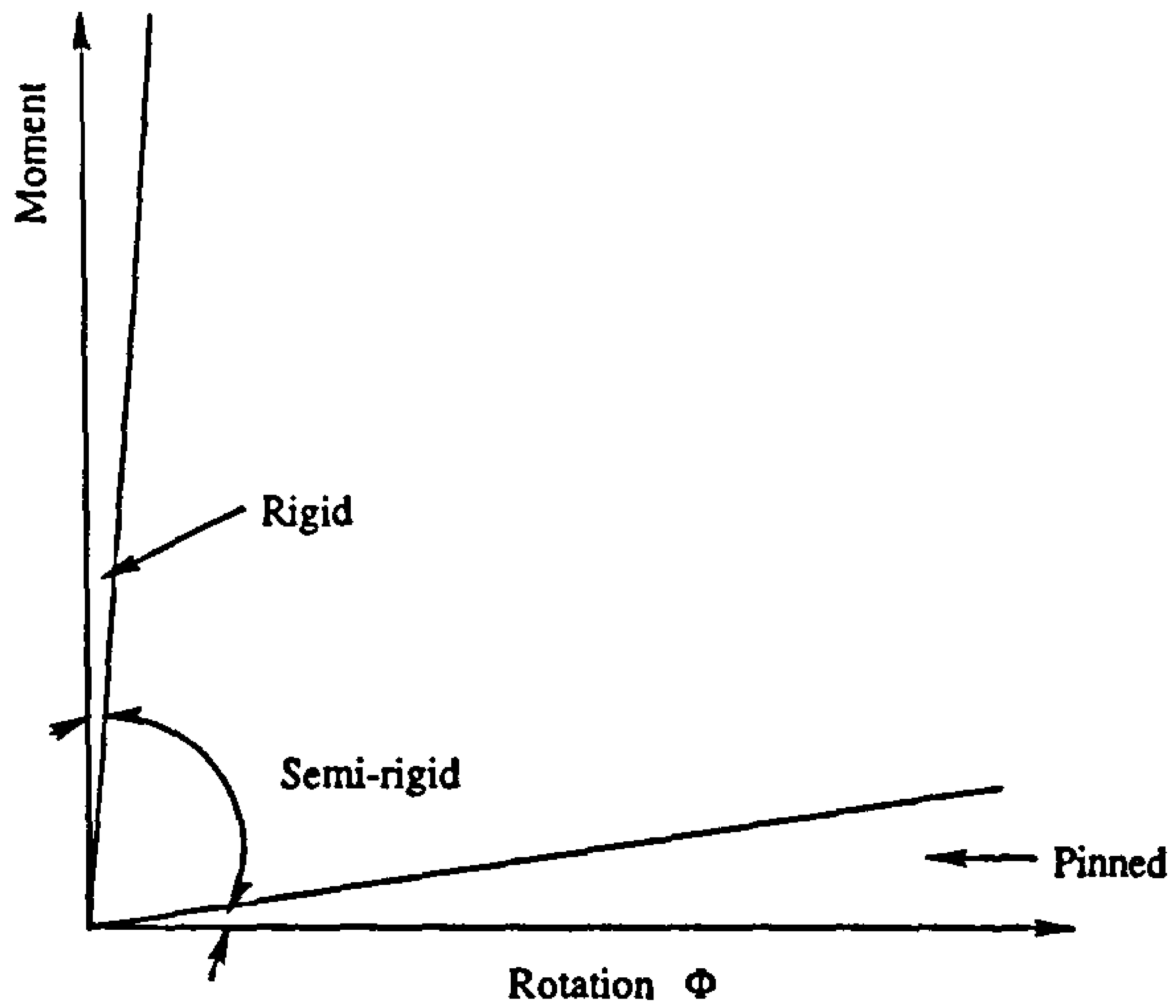


Fig 1.1 Classification of joints

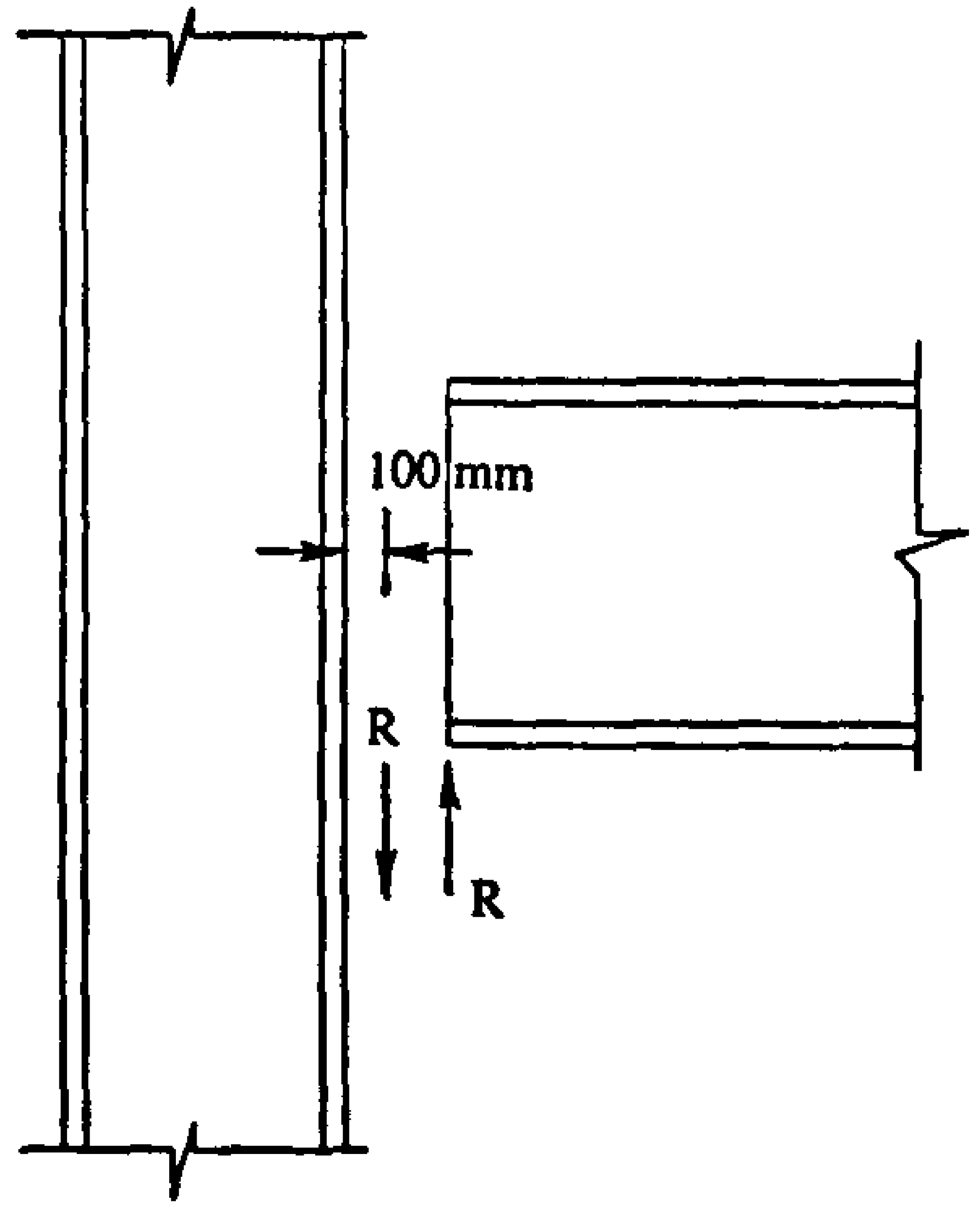


Fig. 1.2 "Eccentricity" moment

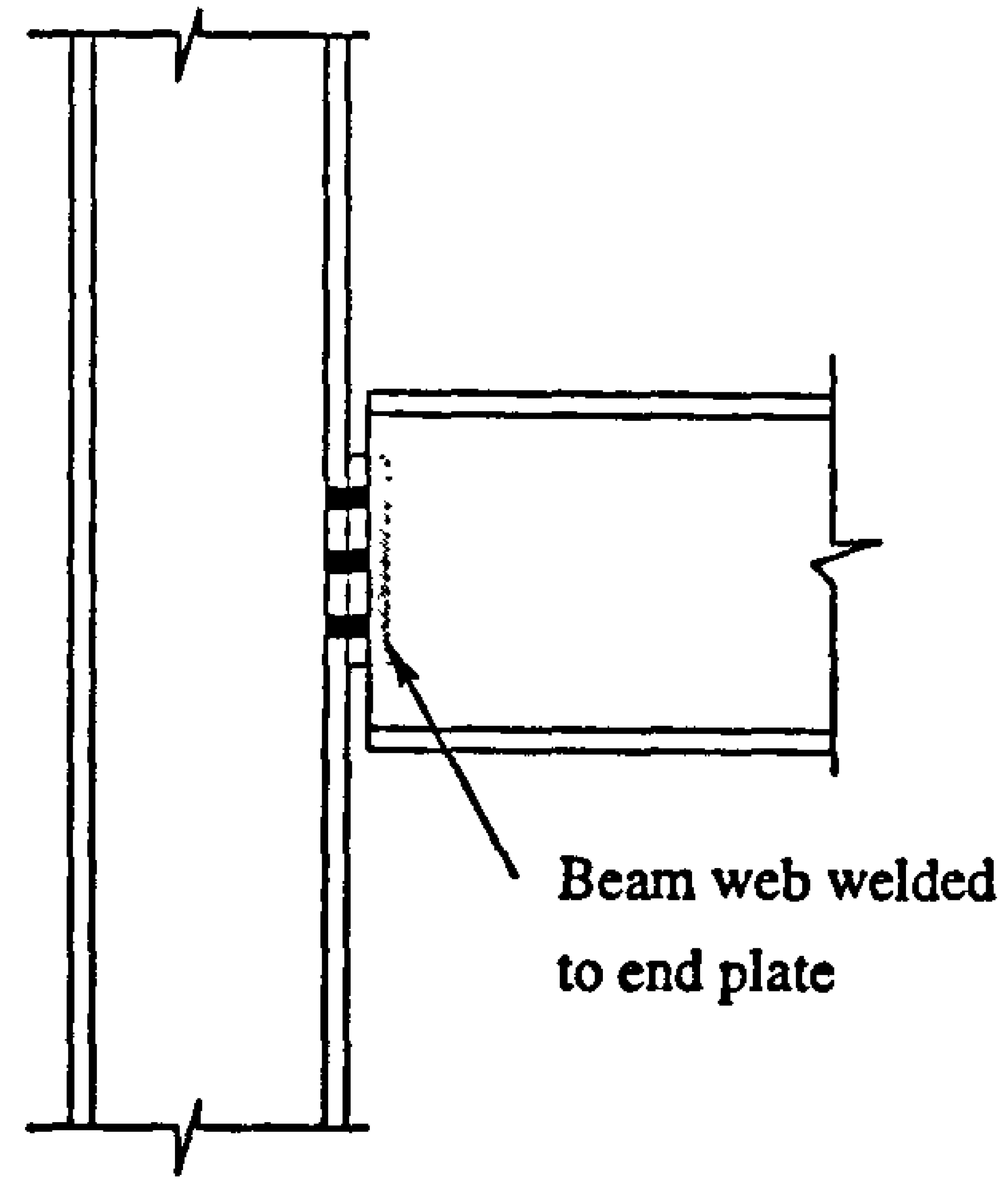


Fig. 1.3 Nominally- pinned joint

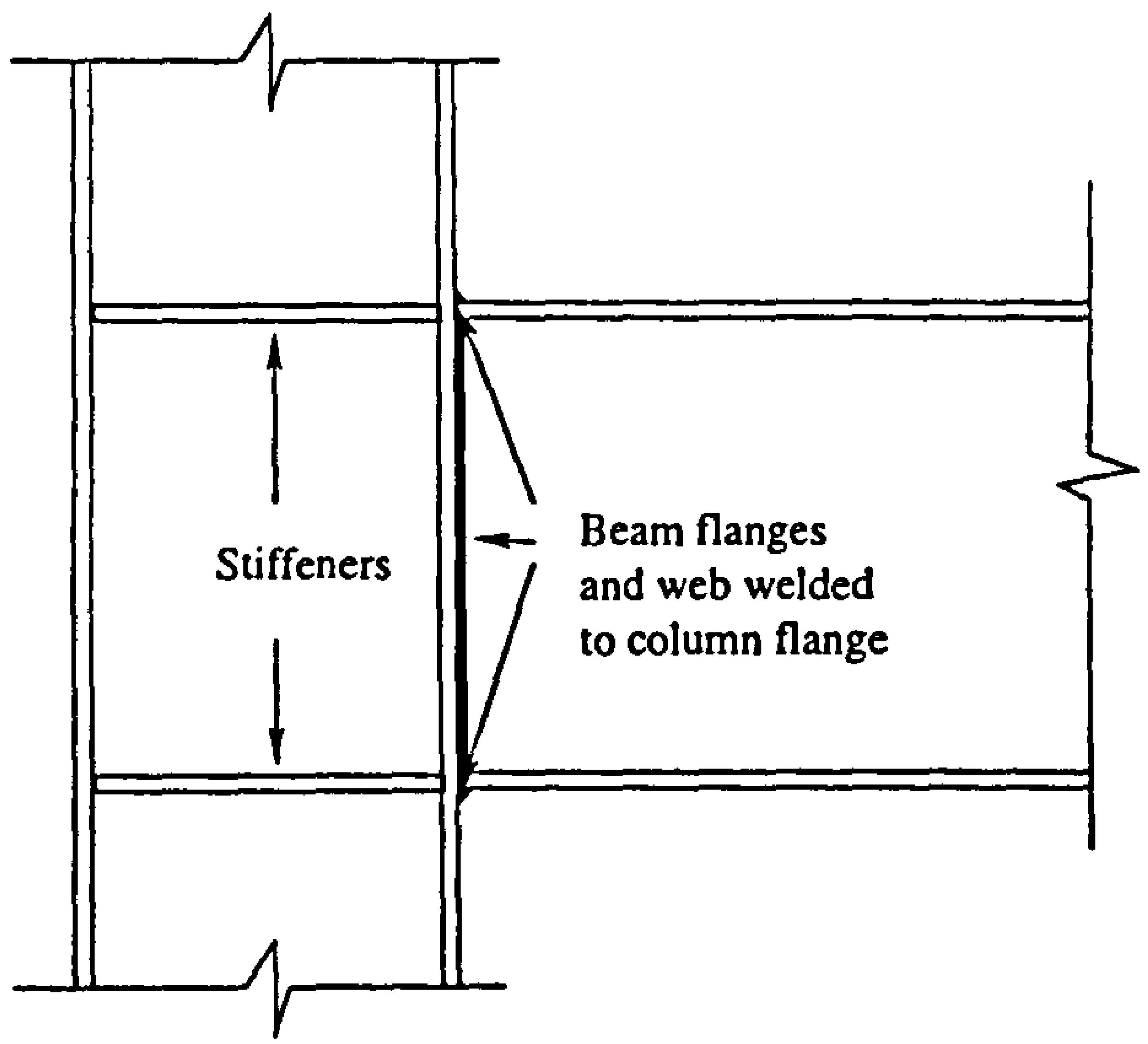


Fig 1.4 Rigid joint

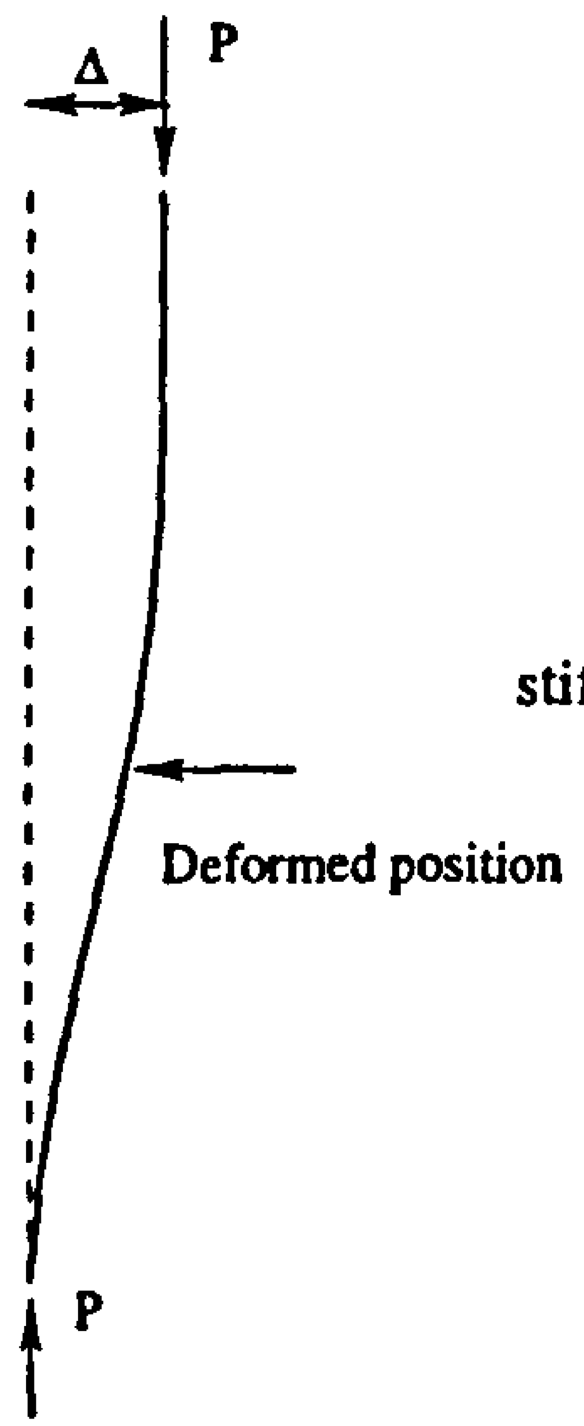


Fig. 1.5 P - Δ effect

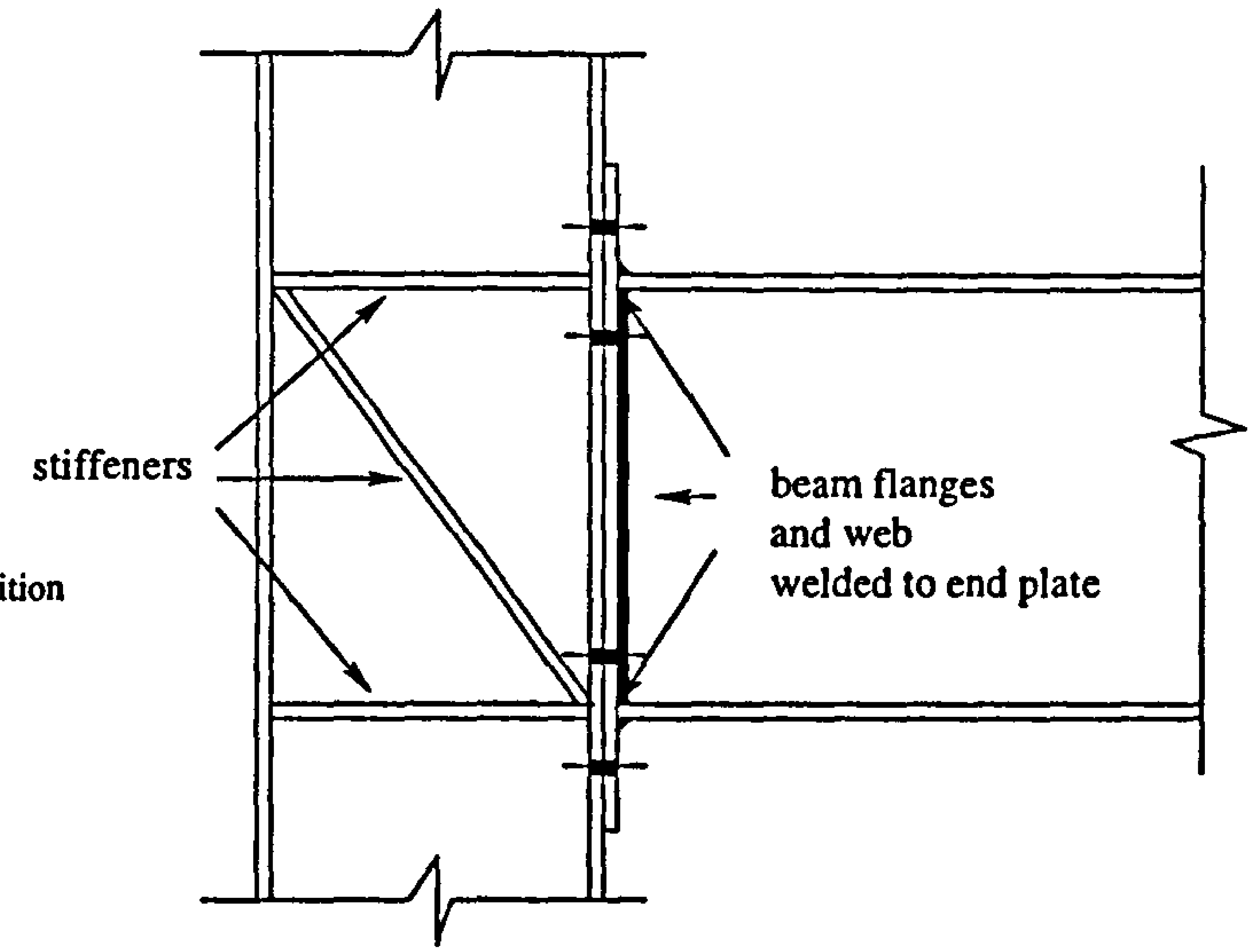


Fig. 1.6 Extended end plate with stiffener

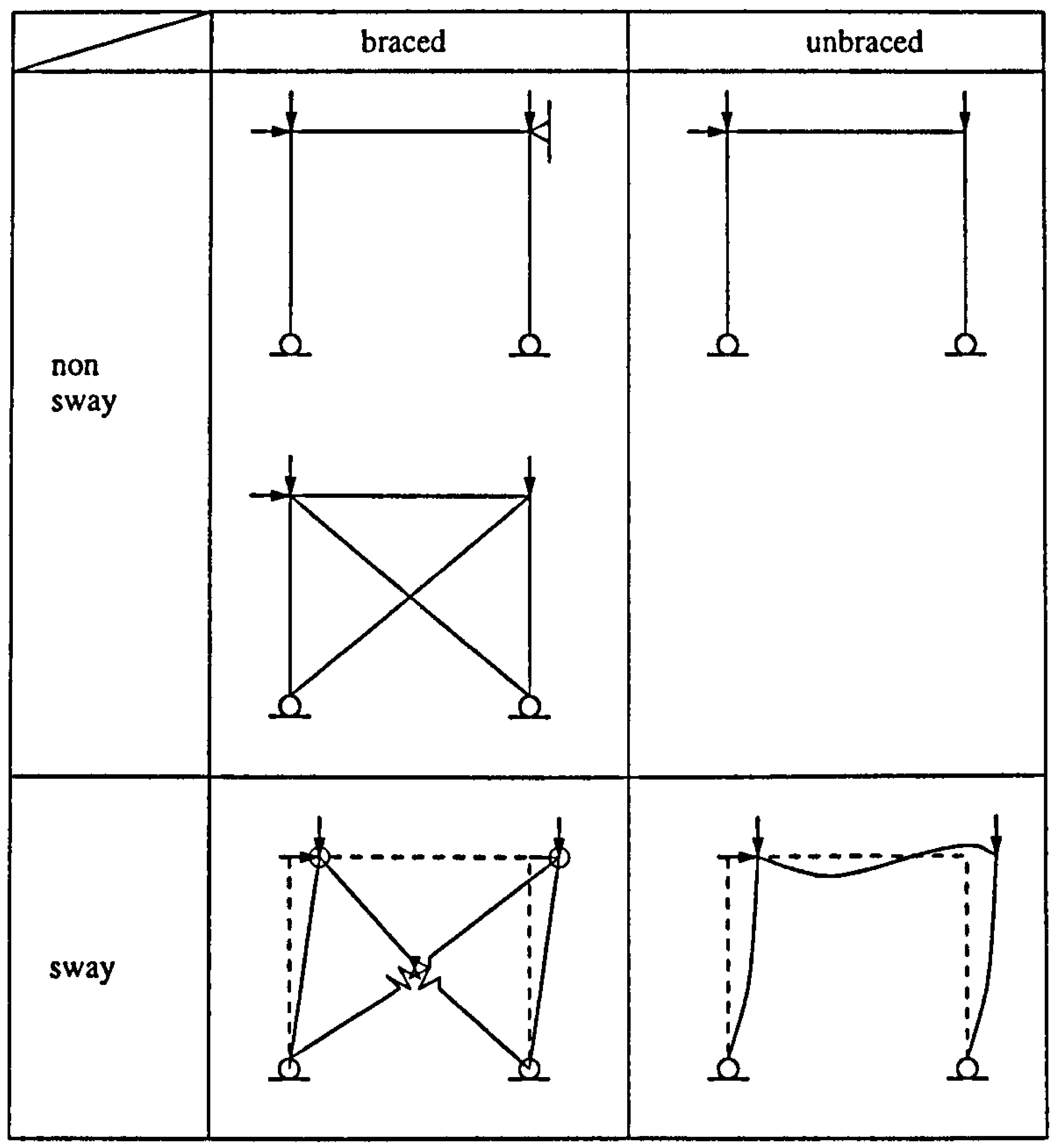


Fig. 1.7 Frame classification

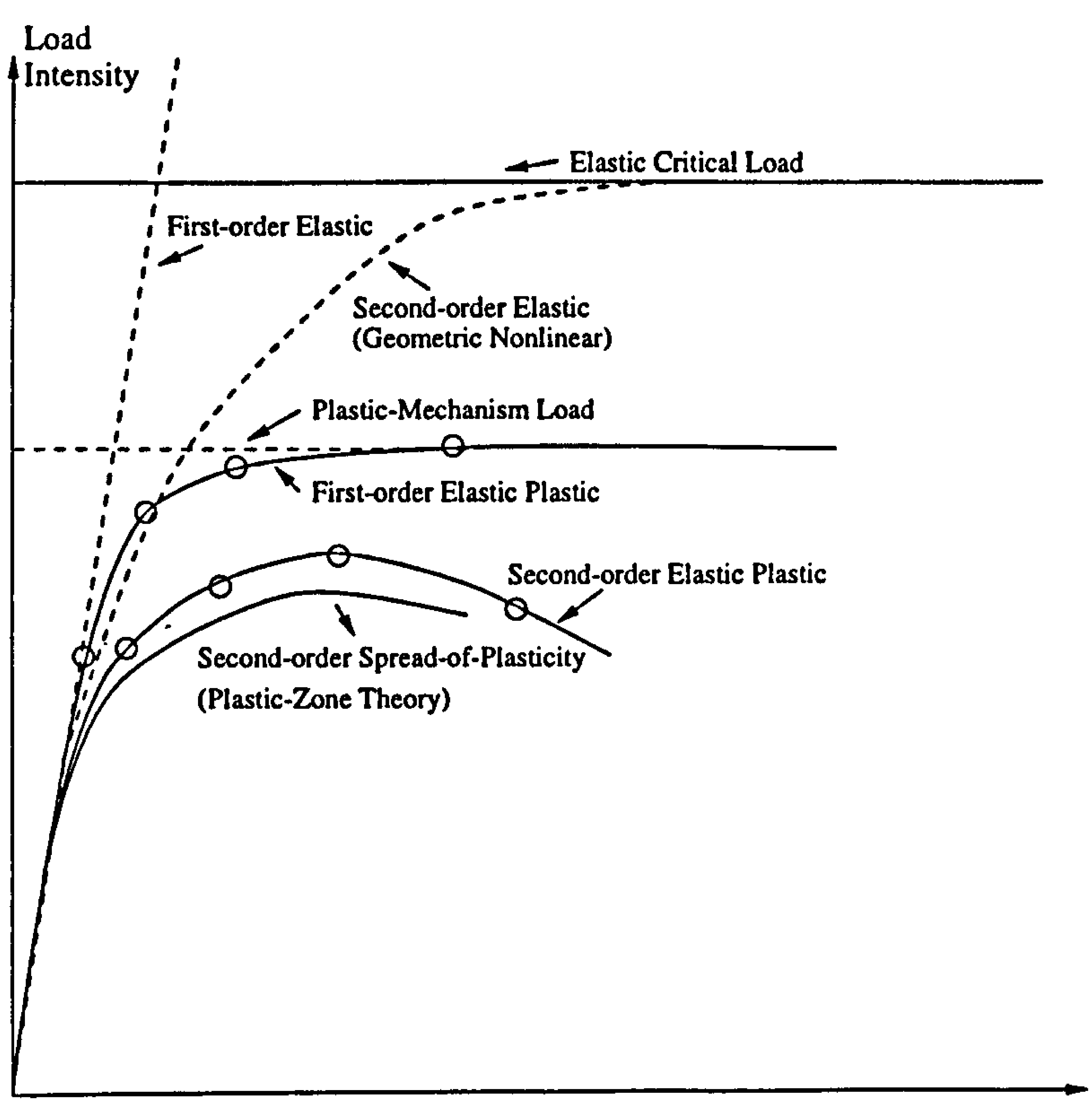


Fig: 1.8 Load-deflection behaviour of plane frame

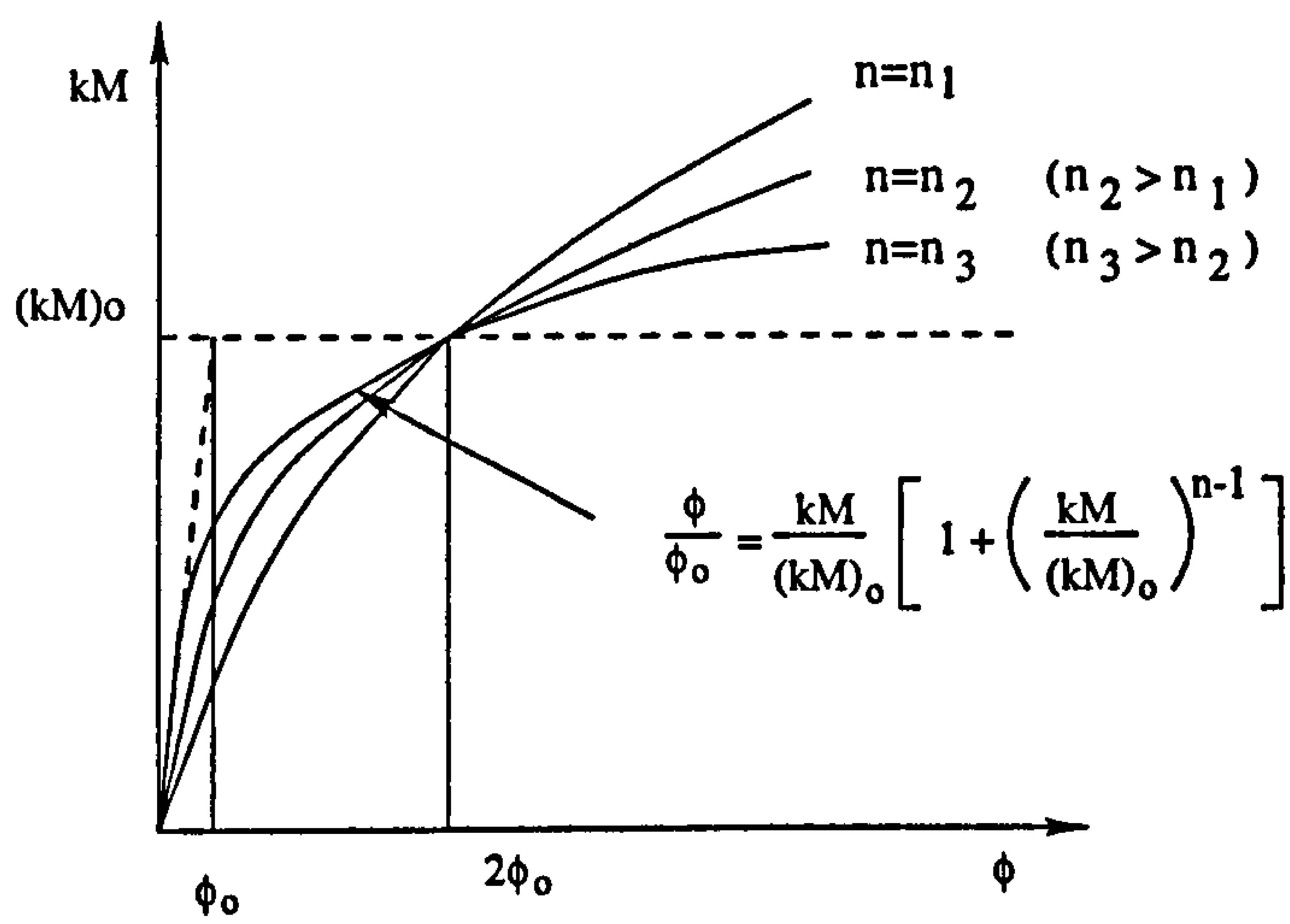


Fig 1.9: Ramberg-Osgood function for moment-rotation curves

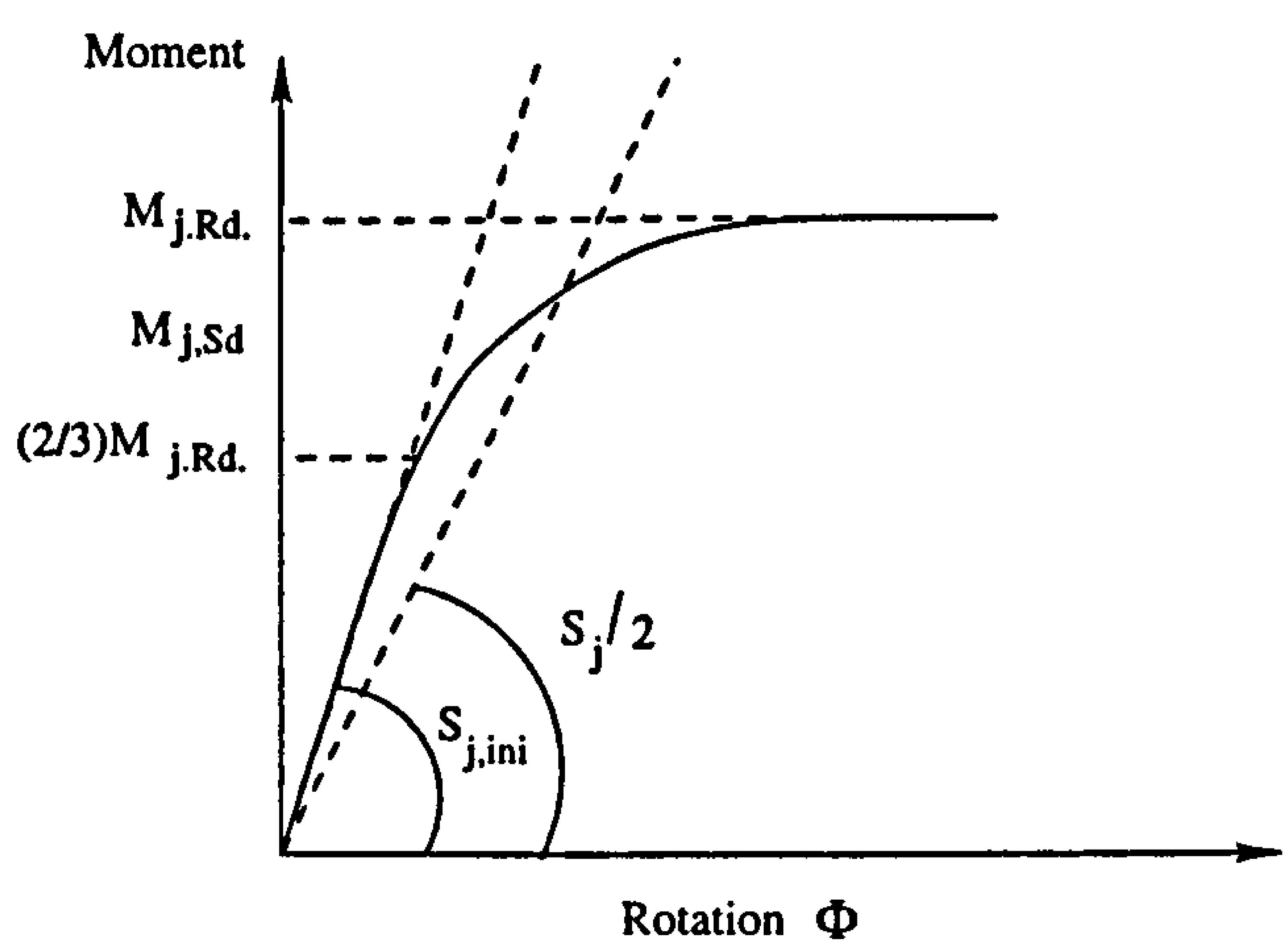


Fig. 1.10 : Linear and non-linear M- Φcurve according to Annex J.

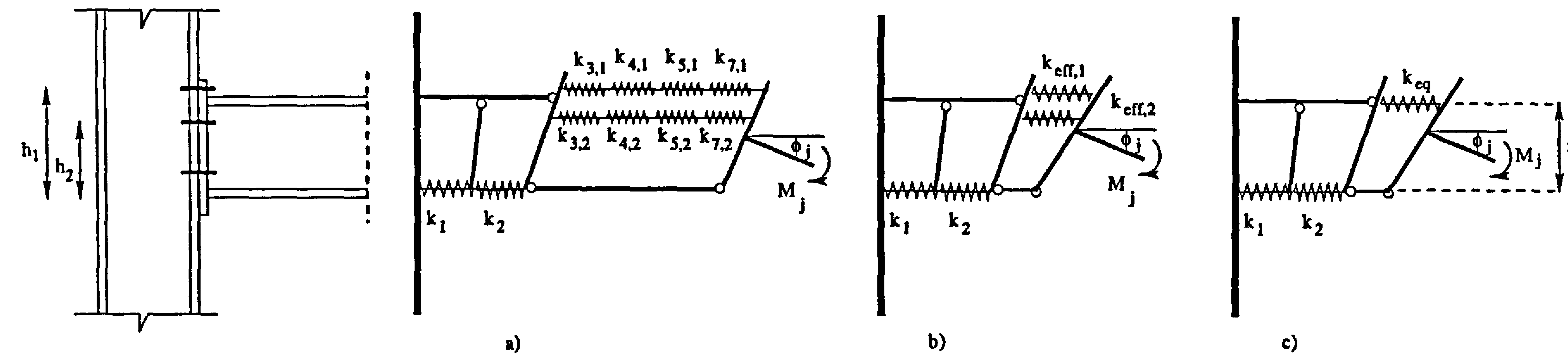
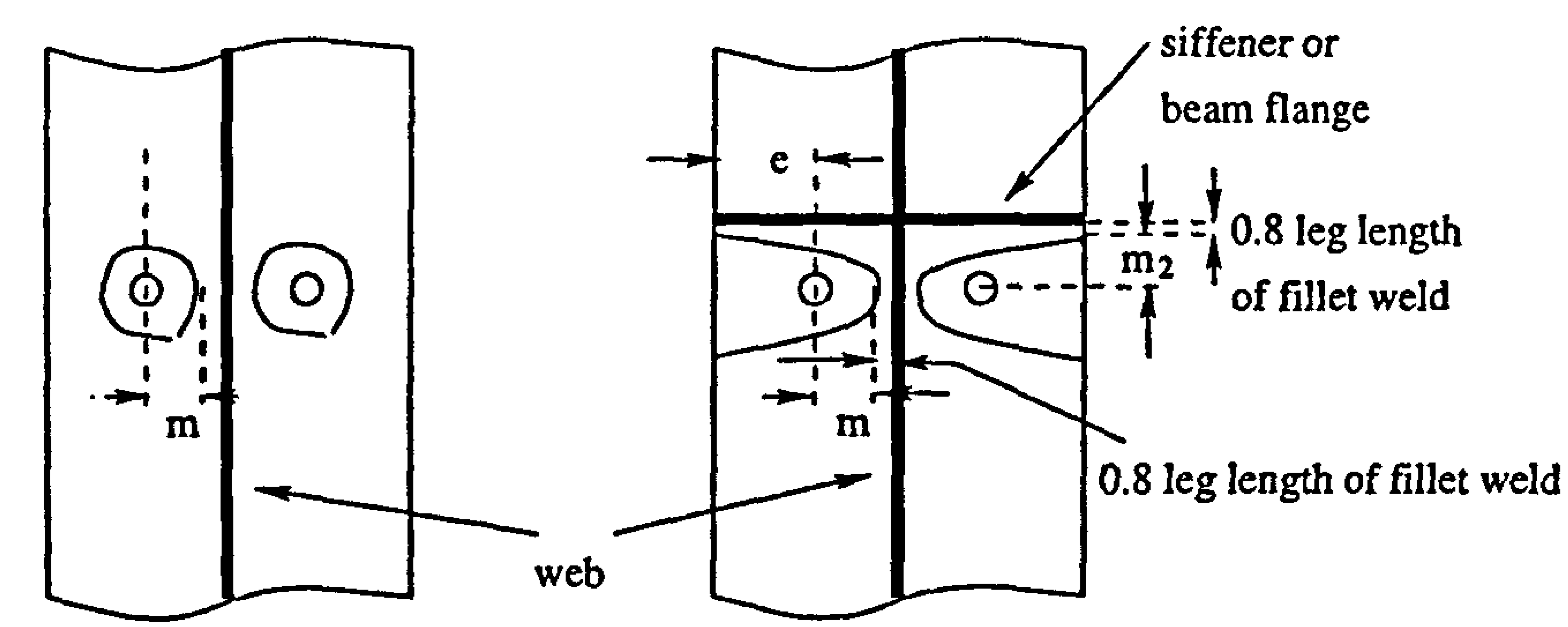


Fig. 1.11 Sring model for a beam-to-column end plated joint with two bolt rows in tension

Spring



a) Hole yielding ($L_{eff} = 2\pi m$) b) side yielding near a stiffener ($L_{eff} = \alpha m$)

Fig. 1.12 L_{eff} for end plate yielding

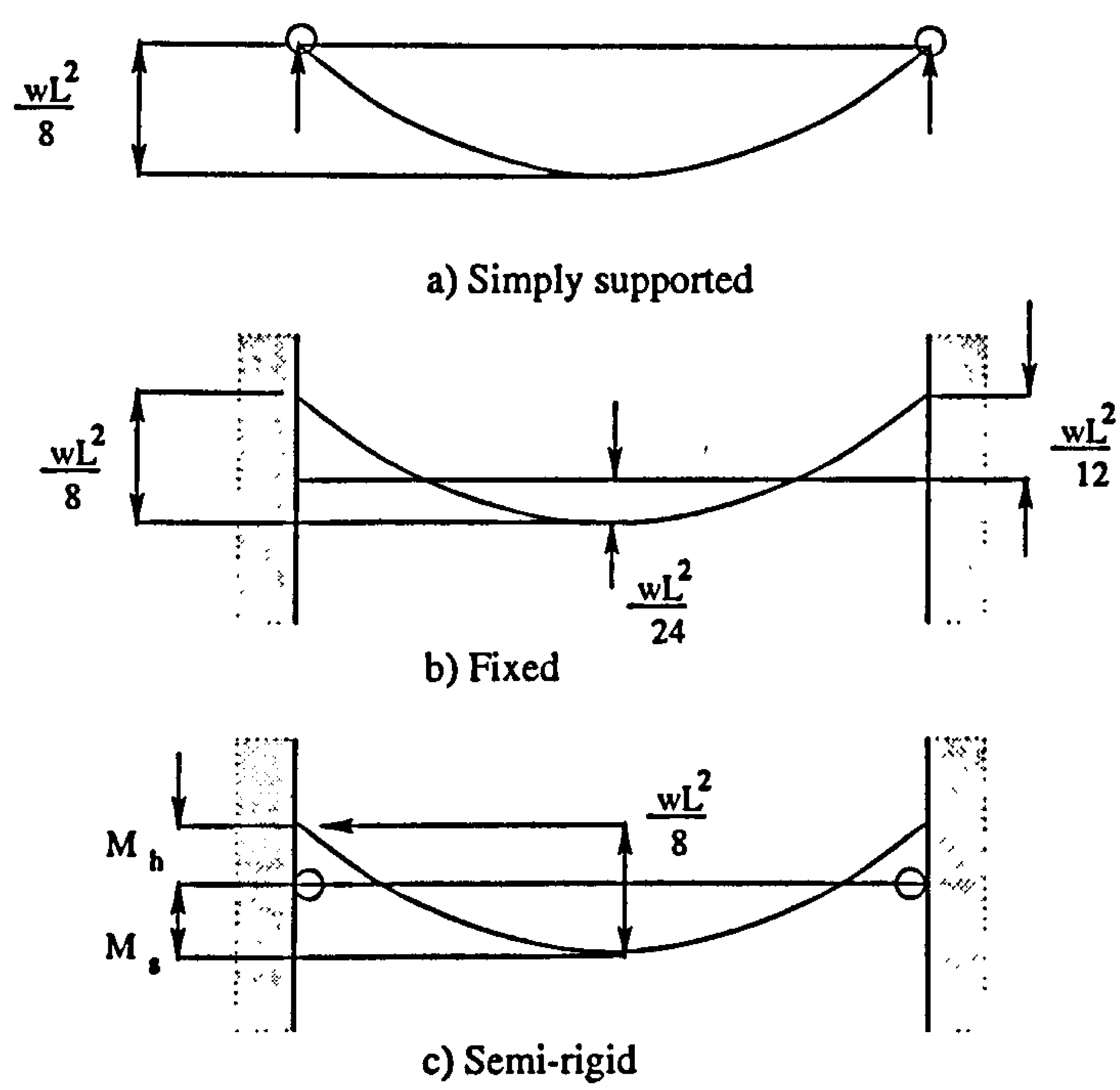


Fig 1.13: Beam with various end condtions.

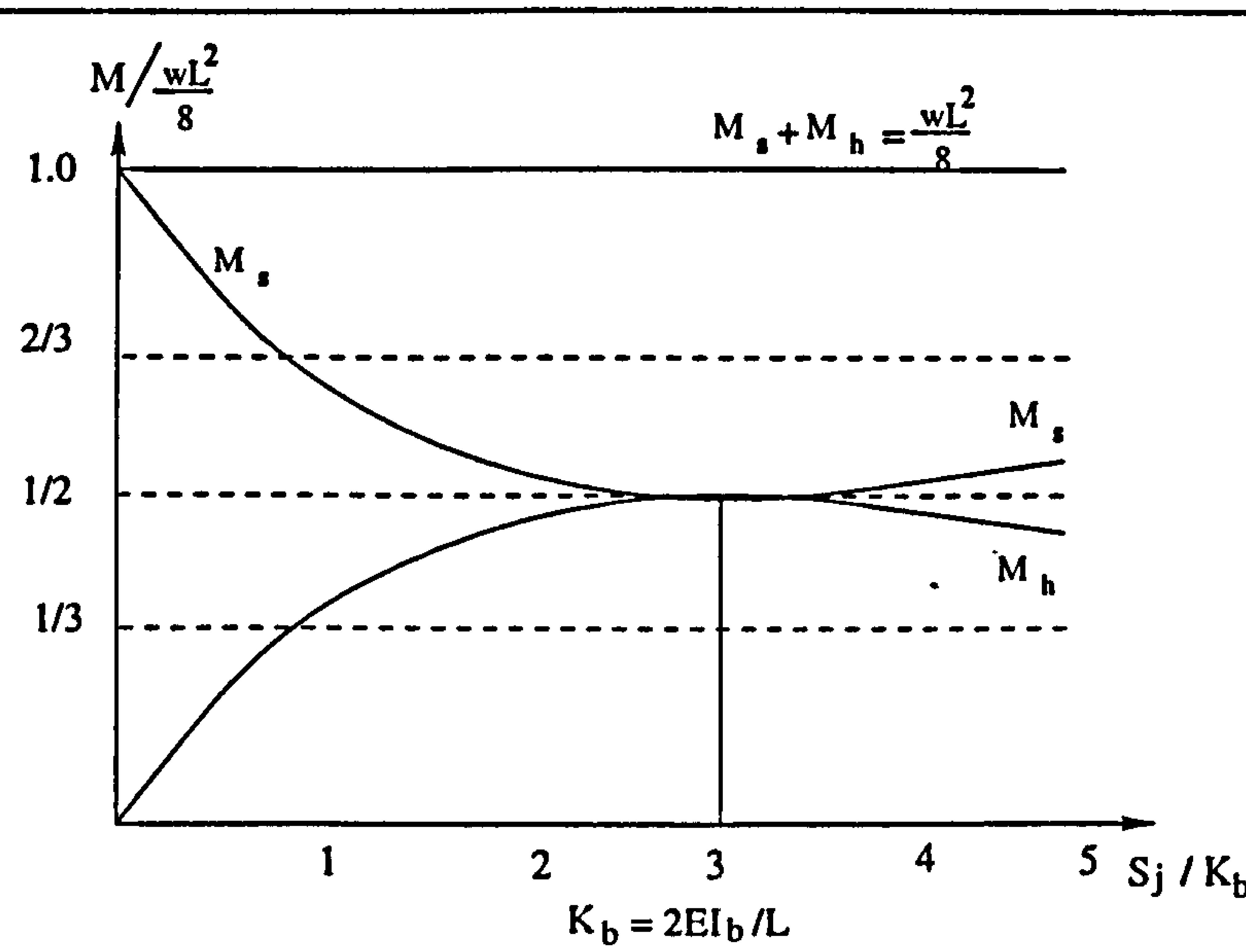


Fig. 1.14: Variation of beam moments with connection stiffness

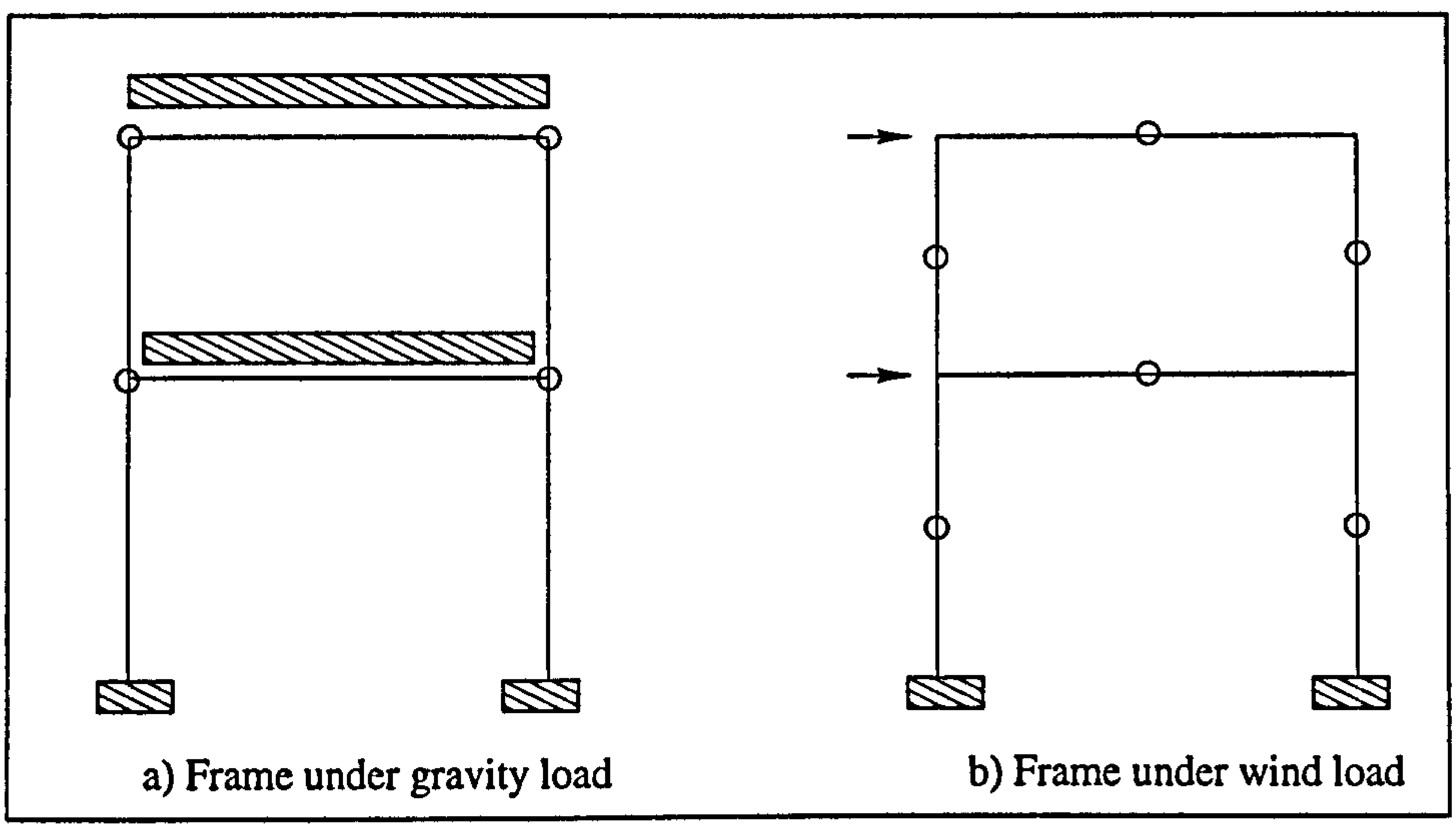


Fig. 1.15 Frame idealisation for "wind-moment" method

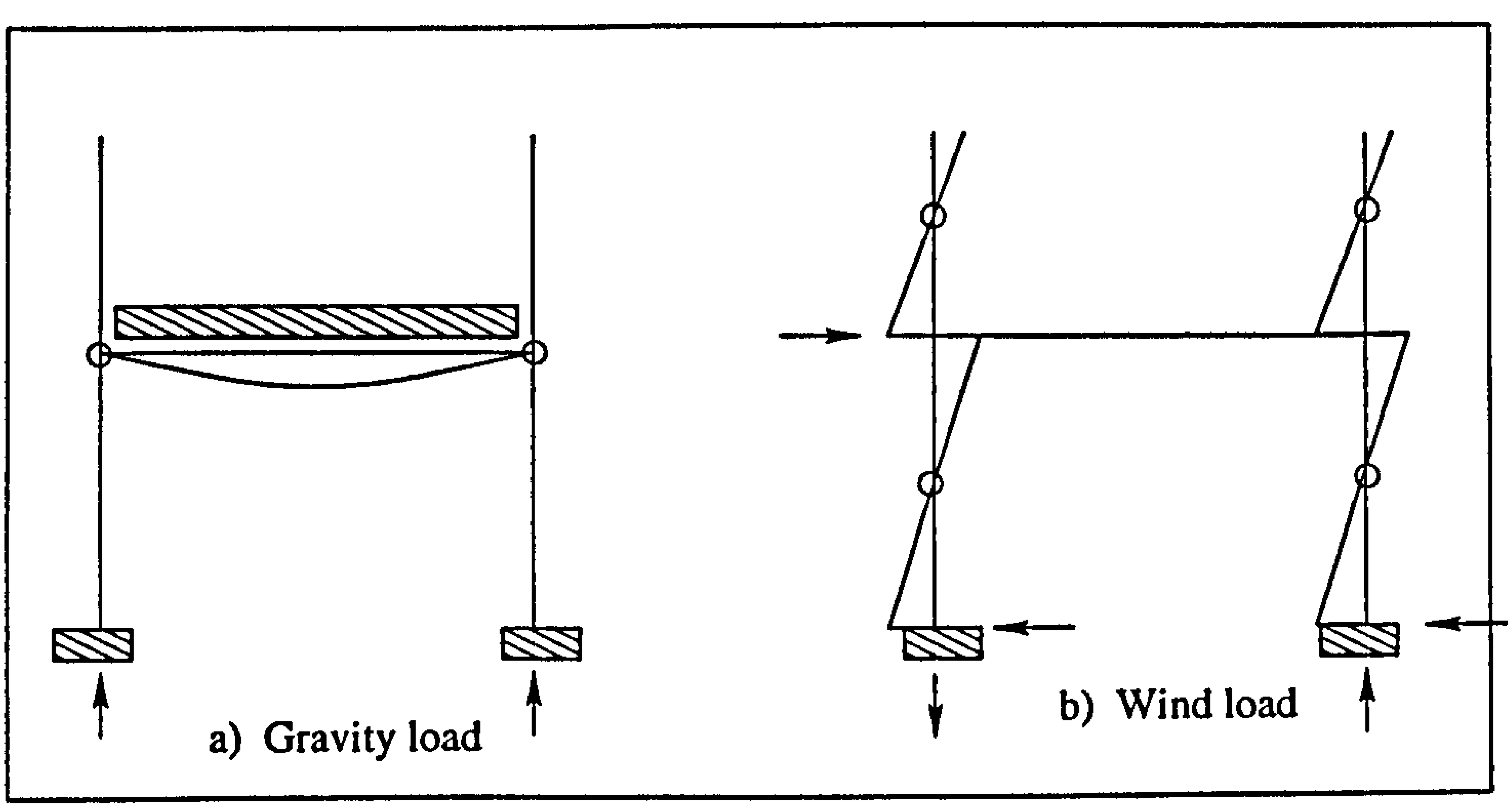


Fig. 1.16 Internal moments and forces according to "wind-moment" method

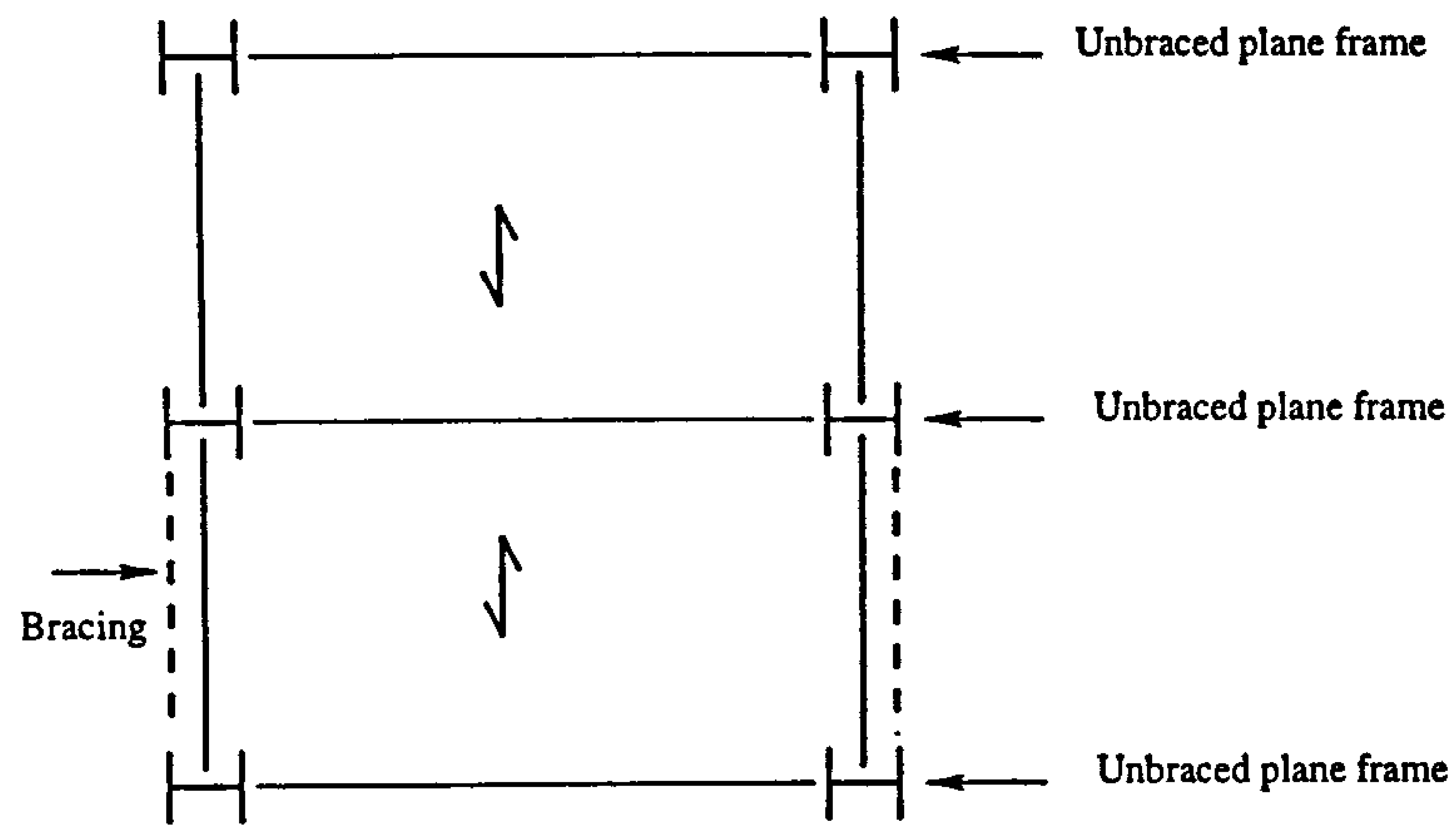


Fig 1.17 Plane frames braced against out-of-plane sway

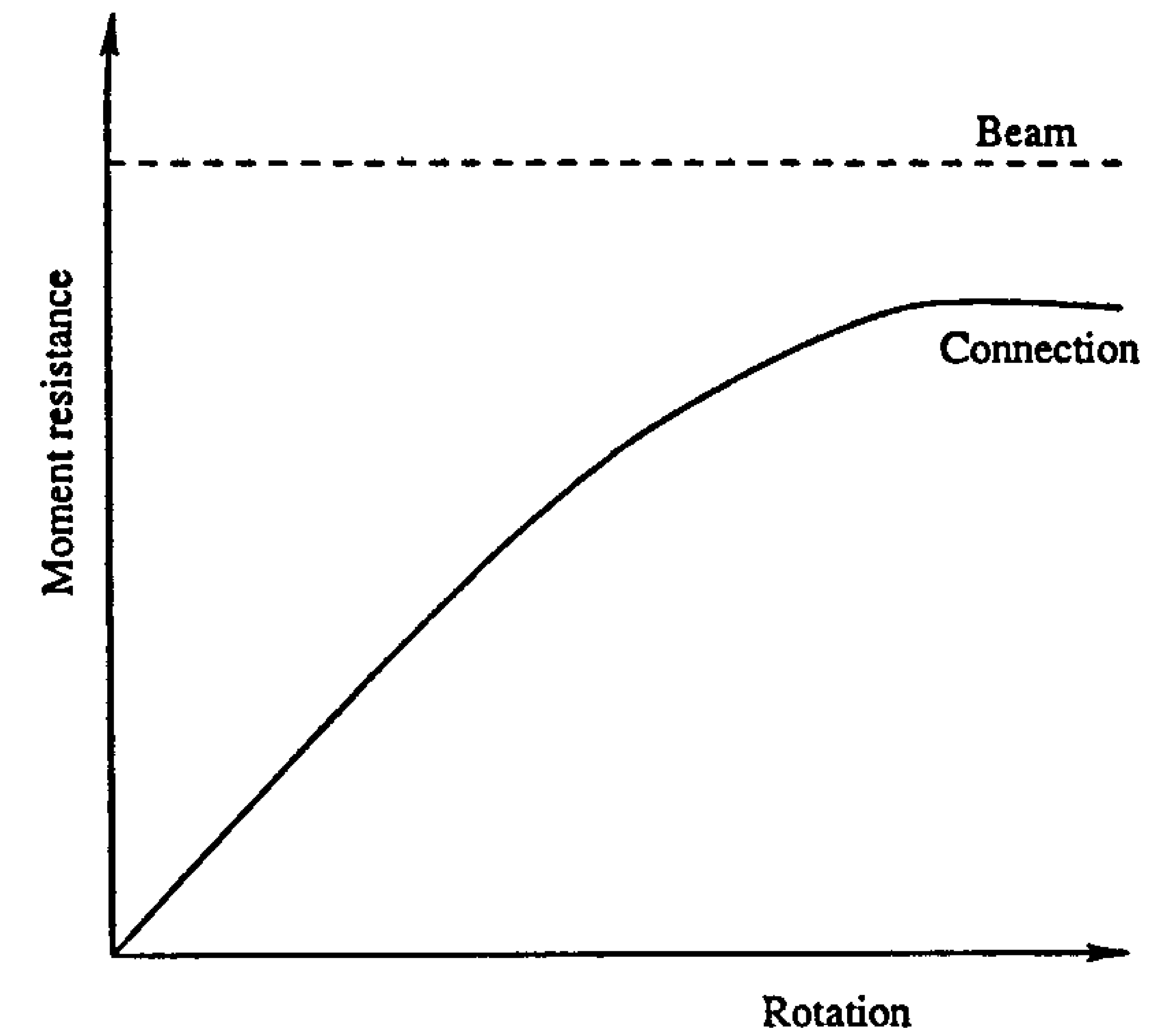


Fig. 1.18 Characteristic of "partial strength" and "semi-rigid" connection

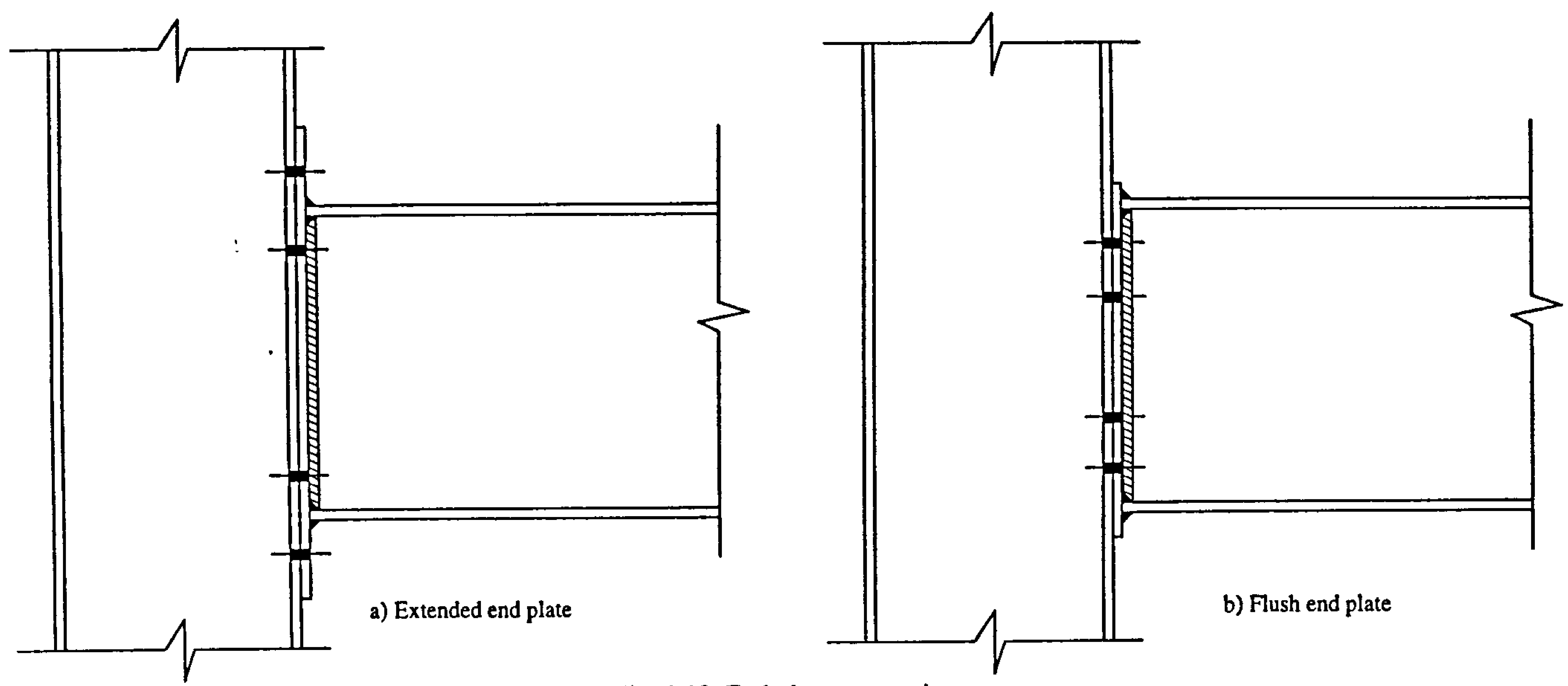


Fig. 1.19 End plate connections

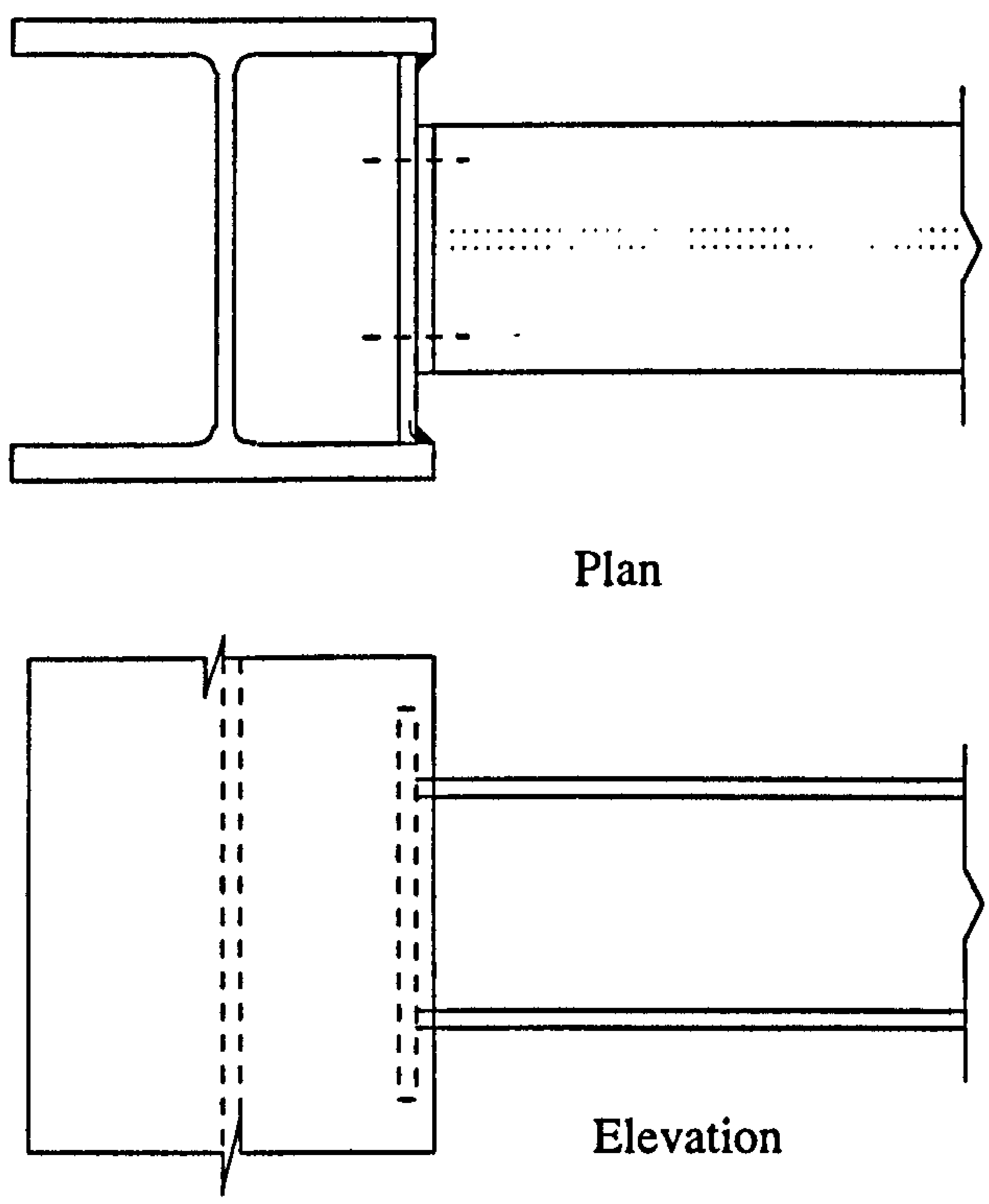


Fig 1.20 Minor axis connections for "Wind-moment" frames

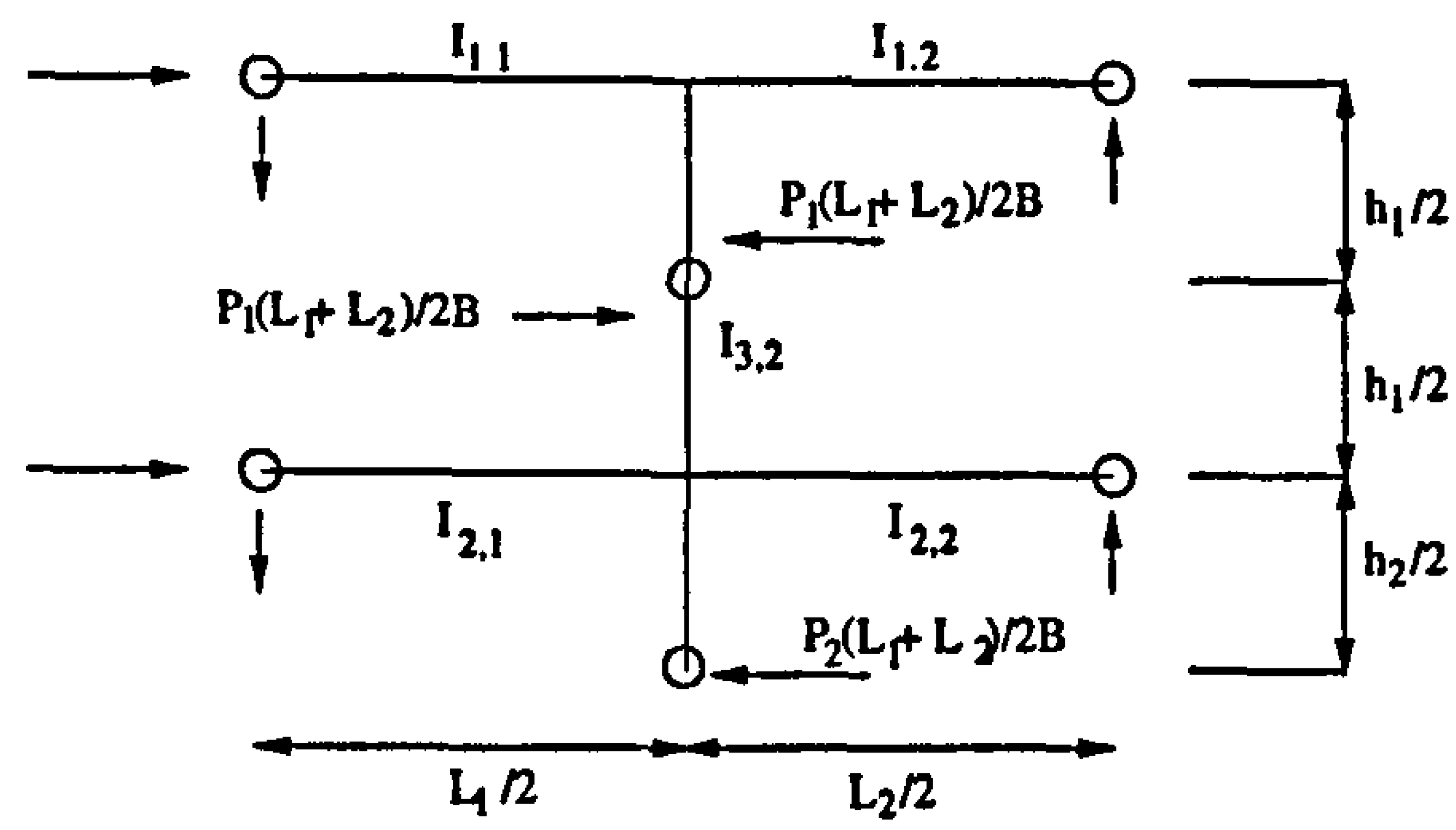


Fig 1.21 Top Storey subassemblage

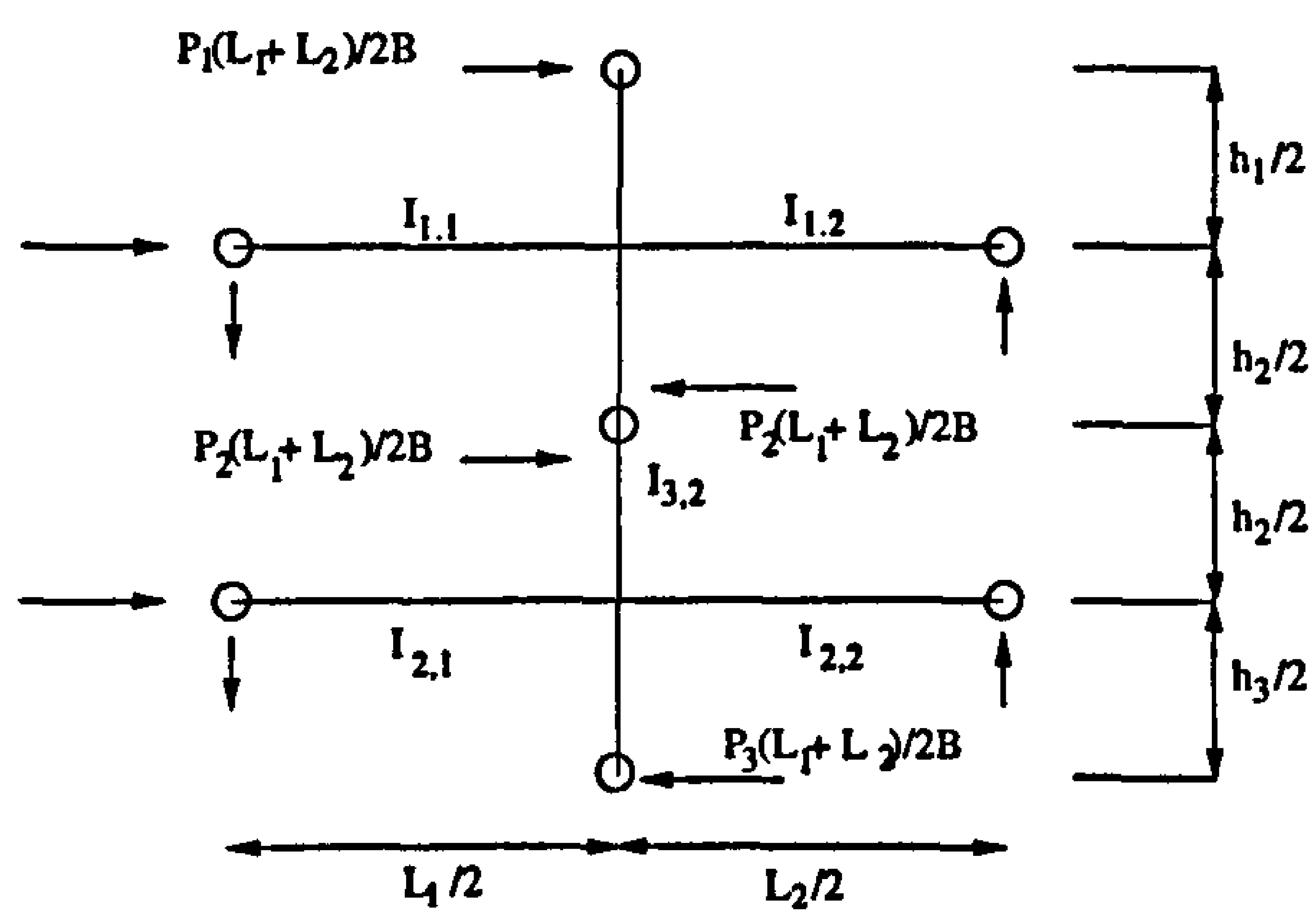


Fig 1.22 Intermediate storey subassemblage

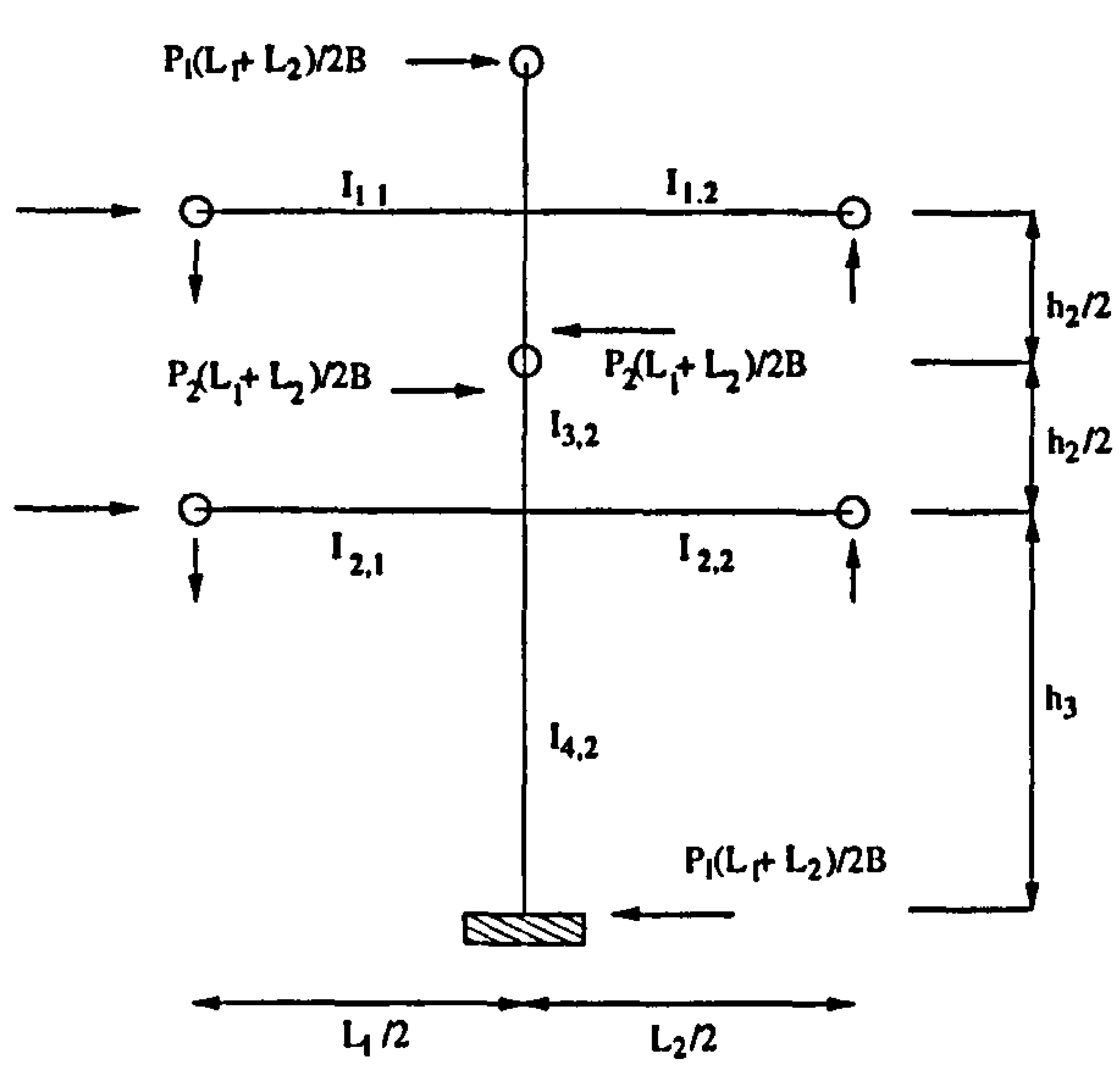


Fig 1.23 Subassemblage for bottom 2 storeys

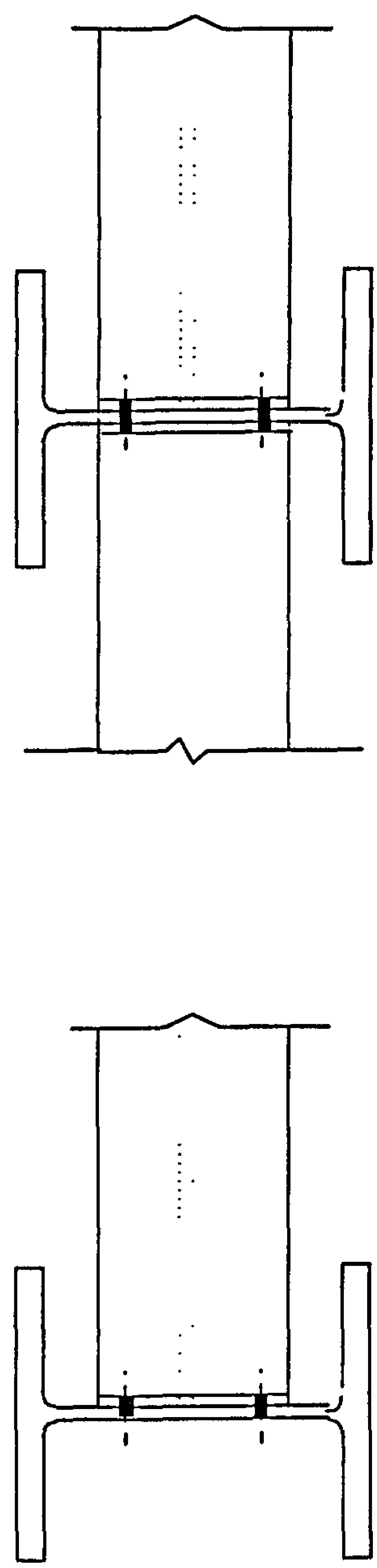


Fig. 1.24 Beam connected to column web on one or both sides

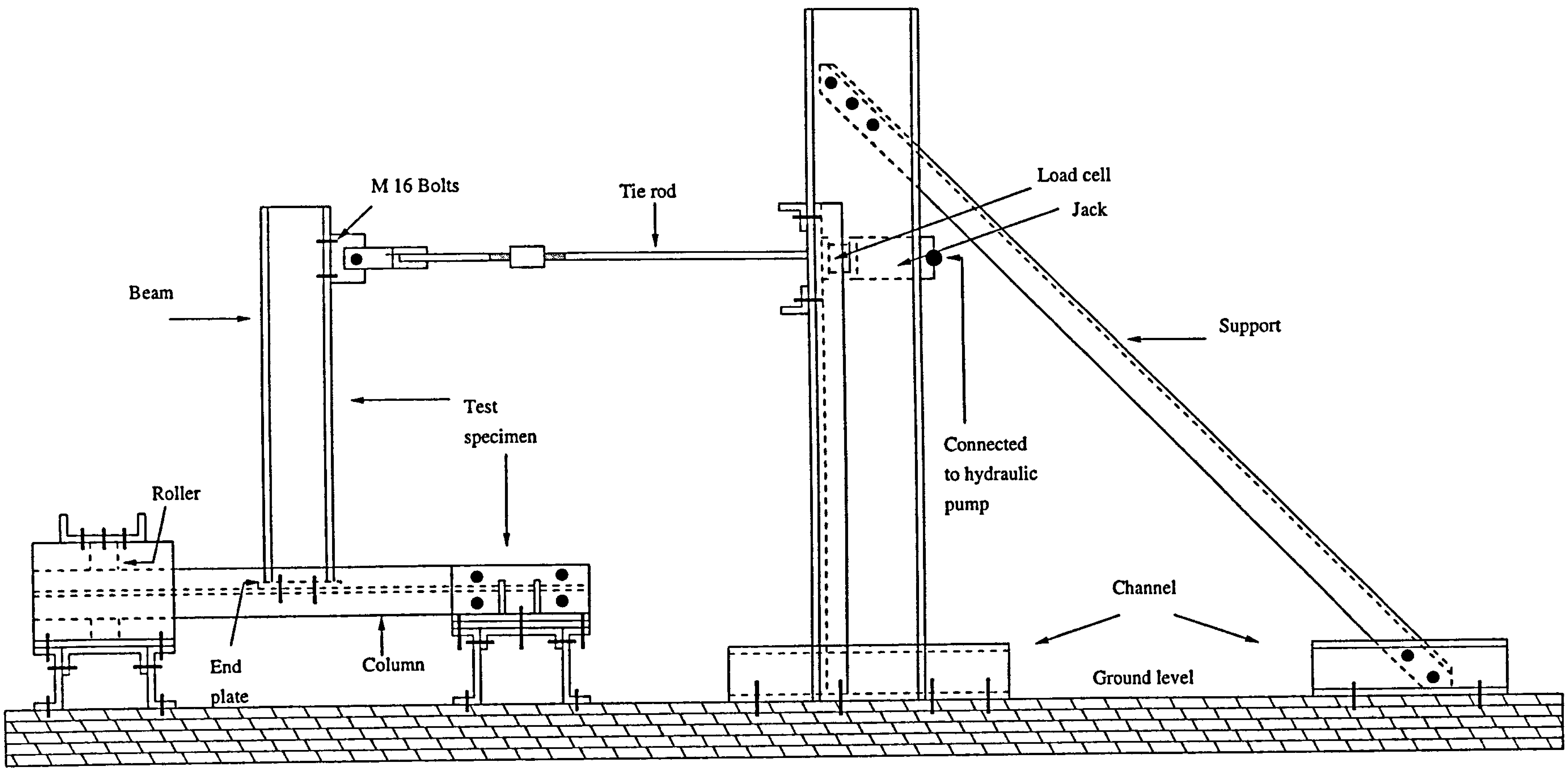
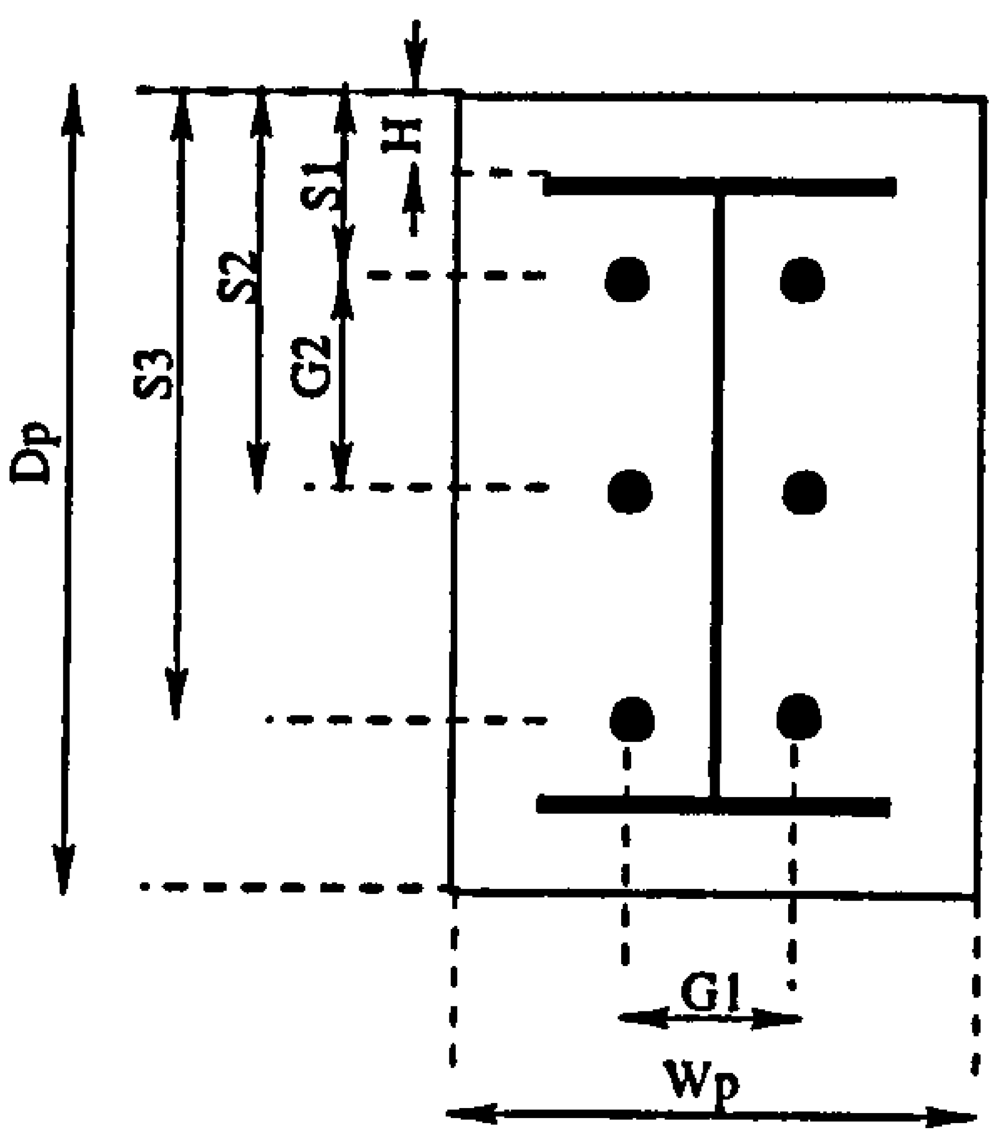


Fig 1.25 General view of test rig



UB	Dp	Wp	H	G1	G2	S1	S2	S3
152x89RJ	170	115	8.8	60	62.4	54	N/A	121
254x102x22	284	120	15	60	60	75	135	209
305x102x25	335	120	15	60	60	75	135	260
305x127x42	335	120	15	60	60	75	135	262

Fig 1.26 Dimension and standard detail for end-plate

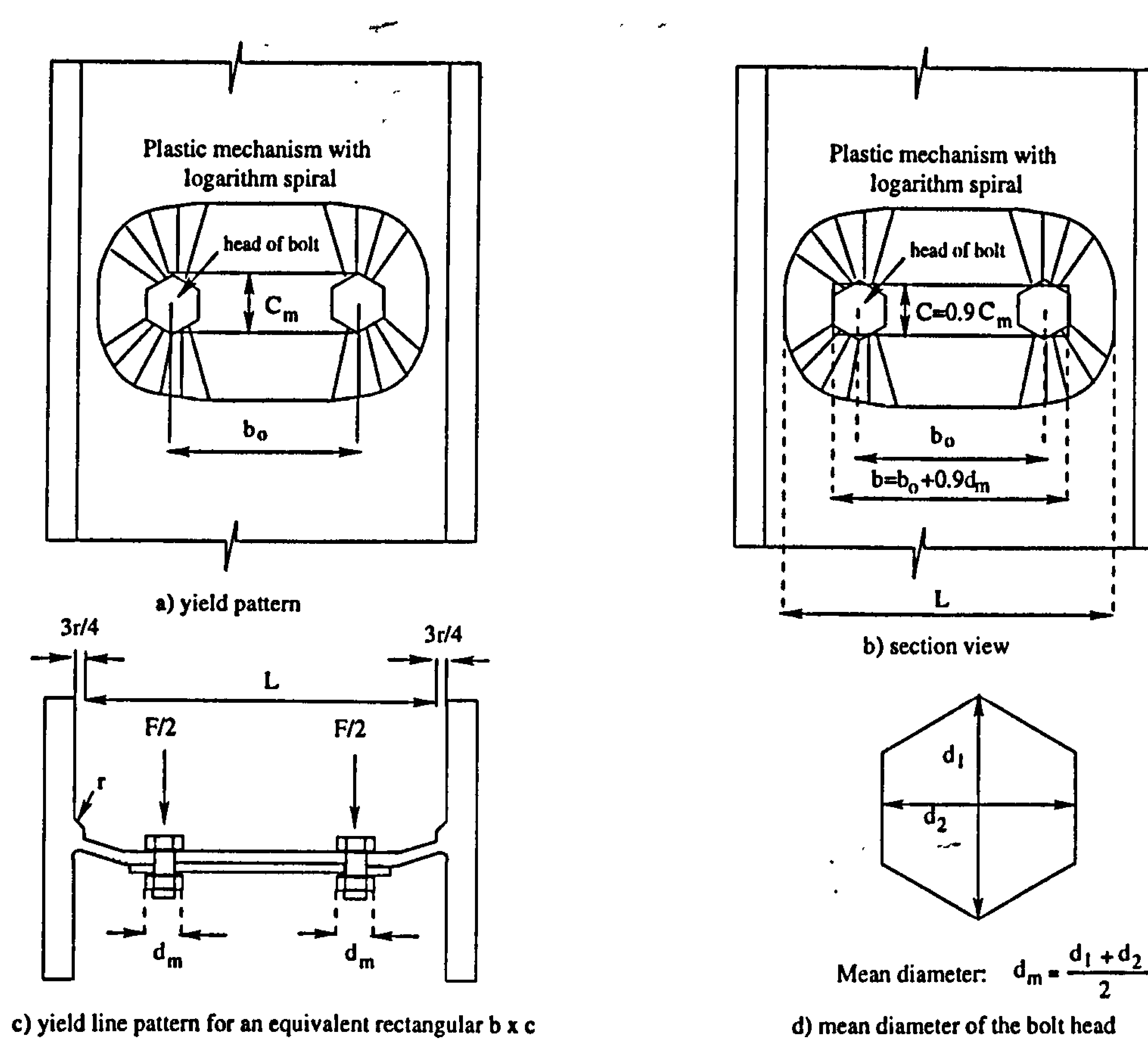


Fig 1.27 Local mechanism for bolted connection.

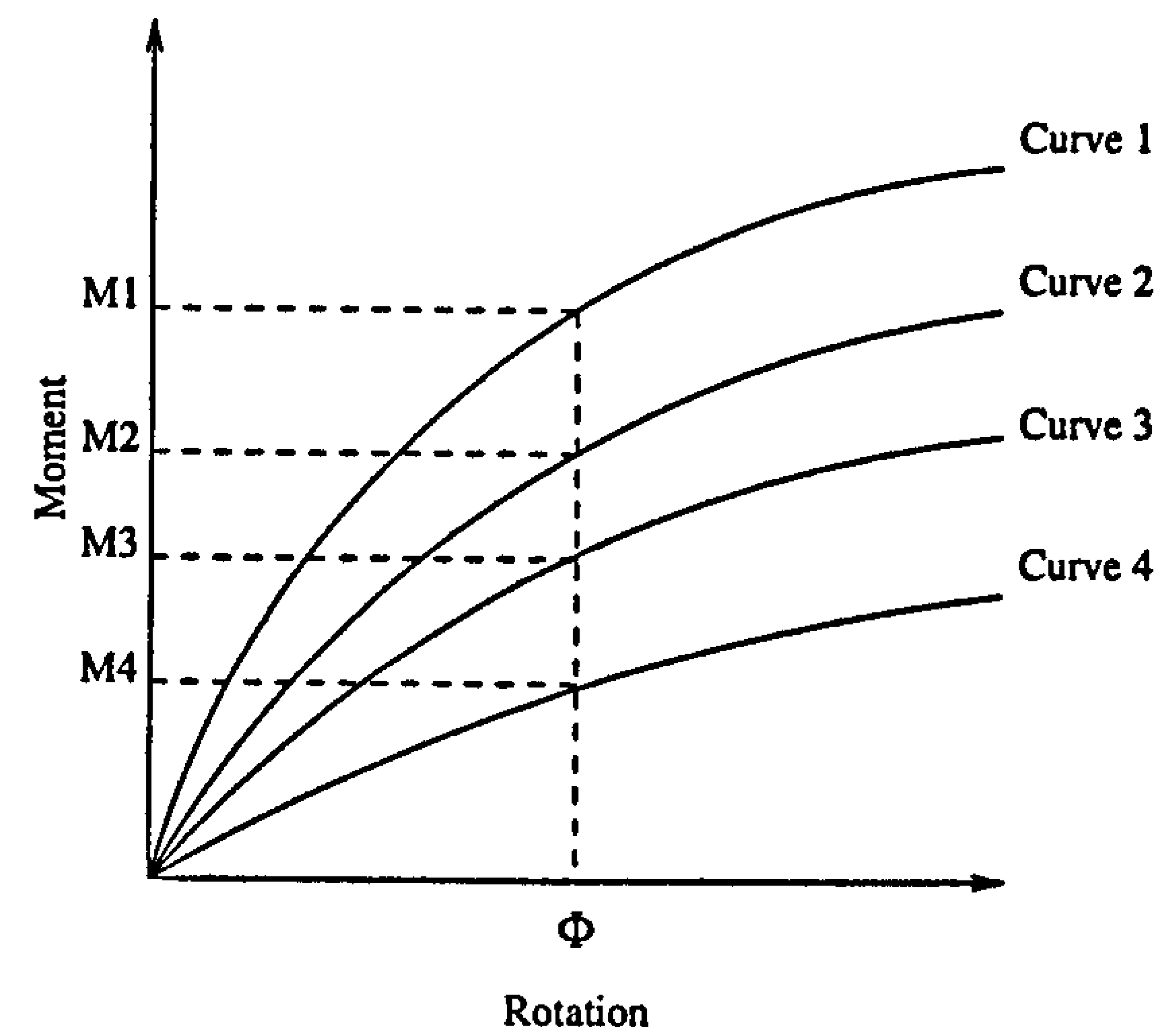


Fig 1.28(a). Moment-rotation curves for non-linear connections differing only in aj parameter at a particular rotation

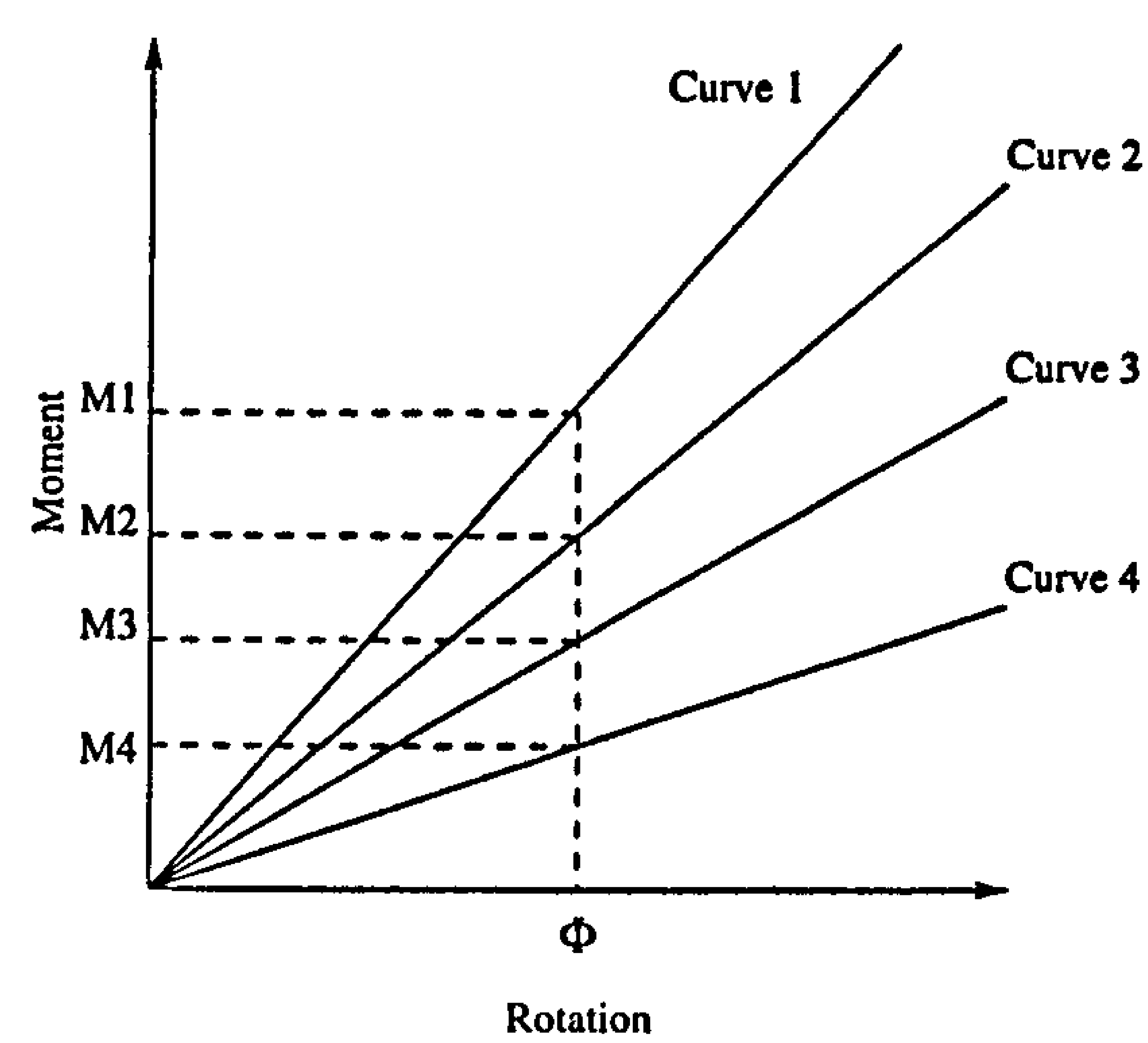


Fig 1.28(b). Moment-rotation curves for linearly elastic connections differing only in aj parameter at a particular rotation

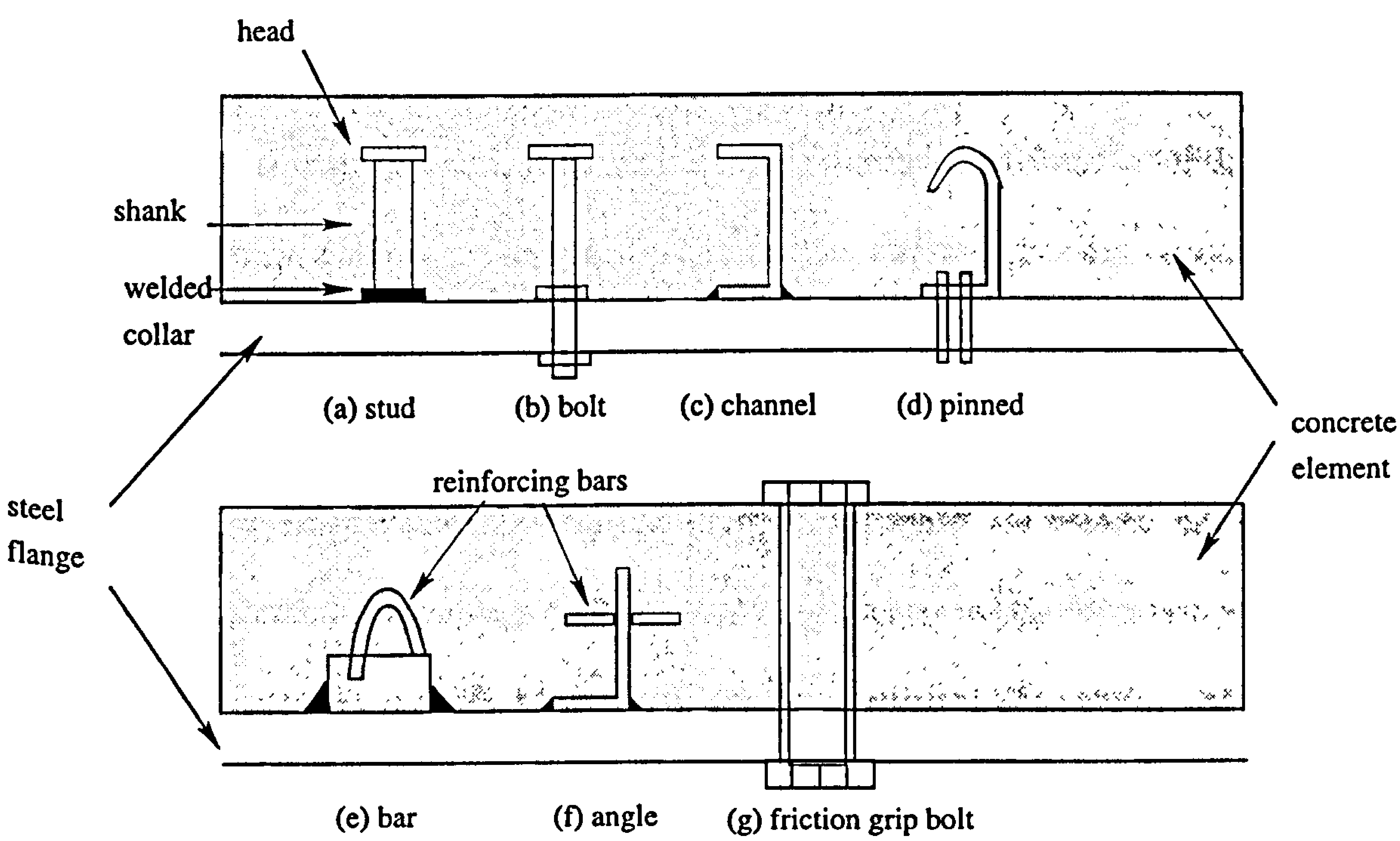


Fig. 1.29: Mechanical shear connectors

Chapter 2

2.0 Economic comparisons between simple and partial-strength design of braced steel frames

2.1 Introduction

Eurocode 3 offers the opportunity to design steel frames as 'semi-continuous' by including the moment resistance of 'partial strength' connections in plastic hinge analysis of the frame[2.1]. This is potentially both an economical and a straightforward method of design, particularly when applied to end plate type connections. The connections are termed 'partial strength' because their moment resistance is less than that of the connected beam. The connections are designed to be ductile, and thus possess adequate rotation capacity to act as plastic hinges. Semi-continuous construction of braced frames is expected to resulted in significant savings in frame weight[2.2][2.3] but the extent of the savings has been questioned[2.4]. The objective of the study was to make economic comparisons in terms of savings of steel weight and savings in cost for braced steel frames, comparing use of pinned beam-to-column connections[2.5] with partial strength joints[2.6]. For the partial-strength approach, the beams in braced frames are designed for a local plastic hinge mechanism taking account of the moment resistance of the connections, taken from standard details. For the design of the columns, the critical load is considered to be when all bays of the braced steel frame are subjected to maximum design loads. All designs were according to the load factors in BS 5950 Part 1[2.7] to determine design loads typical on an office building in the United Kingdom[2.8].

Flush end-plates have been used extensively in the past as nominally pinned joints, even though sometimes designers have specified relatively thick plates. It is well known that such joints possess significant strength and stiffness. However, recent guidance has encouraged the use of partial-depth end plates with web welds only[2.5] as shown in Figure 2.1. With such connections used for "simple" construction, it becomes more difficult to realise significant savings from semi-continuous design, whose connections are expected to be made with full-profile welds. However, the studies still show worthwhile savings in cost, but these will be greater for those fabricators who continue to prefer full-depth end plates in simple joints.

The advantage of the partial strength approach is that it utilises the moment resistance of connections to reduce beam sizes, while avoiding the use of stiffening in the joints. The potential benefits of using this approach can be listed as follows:

- lighter beams
- shallower beams
- greater stiffness
- more robust structure
- lower overall cost.

The disadvantage of the approach is that it may be more expensive to fabricate partial-strength rather than simple connections. Another disadvantage is that the designers need to understand partial-strength behaviour before designing the frame.

2.2 Scope of studies

A series of two-bay and four-bay braced frames, of two, four, six, and eight storeys, was used to compare the two design approaches. The structure was assumed to comprise a series of plane frames at 6 m centres. Floors and roof were assumed to span this distance between the plane frames, and therefore the longitudinal beams were designed only to tie the frames together and to provide lateral restraint to the columns at each floor level. Figure 2.2 shows a general arrangement for a typical plane frame of two bays, within a two-storey structure.

Figure 2.1 and 2.3(a and b) shows typical arrangements for the two contrasting types of connection considered, namely a partial-depth flexible end plate for 'simple' construction, and full-depth end plates known as a flush end plate (see Fig. 2.3(a)) and an extended end-plate (see Fig. 2.3(b)) for the 'semi-continuous' approach. The selection of a flexible end plate is considered appropriate as end plates are the most popular form of simple connection used in the U.K[2.5]. The flexible end plate also provides a very economical form of simple connection, with welding only to the web of the beam. In addition, it is quite versatile for the construction of skew beams and can tolerate reasonable offsets in beam to column joints[2.5]. The selection of flush and extended end plates are considered as they provide the benefit of maximum stiffness of the connection within the ductility constraint[2.6]. The flush and extended end plates also provides a moment resistance in the range 30-50%[2.6] of that of the connected beam. While a flush end plate provides worthwhile resistance, greater resistance is achieved by adopting extended end plate [2.6].

For the form of connection proposed, ductility is assured from full-scale tests[2.9]. Bolts were taken as M20 Grade 8.8. S275 steel was chosen for all end plates. Typical plate thickness used for a partial-depth flexible end plate was 8mm or 10mm whereas a full-depth end plate was 12mm thick. Tables of properties for standard details prepared by Brown[2.10] for the Steel Construction Institute[2.6] were used to select an appropriate partial strength connection. Tables of standard details for the partial depth end-plate connections used as nominally-pinned joints are also available[2.5]. To achieve economy in the semi-continuous design, the columns were unstiffened at the joints, the forces transmitted to the columns being limited by the partial-strength nature of the connections. Beams' spans were taken as 6m and 9m. The column height per storey was fixed at 5m for the bottom storey and 4m for each storey above. Both S275 and S355 steels were considered for the connected members, although S355 schemes were designed for the two-bay and four-bay braced frames of two and six storeys only. Comparisons were made with the aim of:

- designing the beam to have the lightest section and
- designing to achieve the shallowest section.

2.2.1 Loading

2.2.1.1 Loading on beams

Dead load

The load was derived from :-

150 mm thick precast floors[2.11] = 2.46 kN/m^2

Finishes[2.11] = 1.5 kN/m^2

Self weight for steel beam estimated at 0.8 kN/m = 0.13 kN/m^2

Total dead load = 4.09 kN/m^2, say 4.00 kN/m^2

Imposed load

For floor (including partitions) = 4 kN/m^2

For roof = 1.5 kN/m^2

Reduction in live load is made when a column supported more than one level, according to BS 6399[2.8].

2.2.1.2 Loading on columns

Self weight was approximated to be 5 kN/storey. According to Bates[2.12], 225 mm thick brick cladding plastered on one side is approximately 4.8 kN/m^2. For 1.5 m high wall, the total weight due to this cladding is therefore equal to 43 kN. Including glazing, the value was increased to 50 kN, which is in accordance with the earlier SCI studies on savings in weight[2.13]. Self weight for minor axis beams was taken to be 25 kg/m or 2.0 kN for 6m frame. Load from brickwork and glazing will only apply to the external column. A parapet 0.75 m high was assumed at roof level.

2.2.2 Design approach

The studies described in this chapter are concerned with costs associated with fabrication in the U.K, and use has therefore been made of British design rules (BS 5950, 1990) [2.7], and resistance tables for joints (Steel Construction Institute)[2.5][2.6]. The latter are based on Eurocode 3[2.1] as a design model but with strength checks modified to suit BS 5950[2.7]. Computer software was prepared by the author to analyse and design for both simple and semi-continuous construction. Software written by Reading[2.14] for wind-moment design was adopted and modified to suit the study of braced frames.

2.2.2.1 Simple Construction

This followed usual practice according to BS 5950[2.7]. Hence, although the connections were designed for shear only, external columns were designed for a nominal moment due to an assumed eccentricity in the application of beam end reactions. This was taken as 100mm from the face of the column. If a beam was not a roof beam, the moment was divided equally between the columns above and below (clause 4.7.7). The effective length for both major and minor axes was taken to be 0.85L. The conditions to determine the effective lengths are given in Table 24 of BS 5950 Part 1[2.7]. All beams were subjected to uniformly distributed load, and the design moment in simple construction was therefore $wL^2/8$. The effective span for beams in simple construction was taken from centre of column to centre of column. The beams were assumed to be fully restrained against lateral buckling. The connections were flexible end plates selected to resist the design shears[2.5]. In the tables provided[2.5] checks are made on :

- bolt capacity
- end plate shear and bearing
- beam web
- beam-to-plate weld

- column flange or weld.

The buckling resistance moment M_b, of the column was calculated using $\lambda_{LT}=0.5L/r_y$ (clause 4.7.7 of BS 5950)[2.7]. The value of buckling resistance was then calculated using formulae in Appendix B of BS 5950; the axial resistance P_c, was calculated from the formulae in Appendix C of BS 5950, assuming that the sections were Universal Columns of thickness not greater than 40 mm. The selection of column sections were based on minimum depth and over the height of the structure the same nominal size was maintained as far as possible. Column splices were provided every two or three storeys as recommended by SCI[2.5] and were located 500 mm above floor level. According to the code, the following stability check needs to be satisfied in "simple" construction:-

$$\frac{F}{P_c}+\frac{M}{M_b}\leq 1.0$$

where M is the maximum end moment in the column.

The design strength for steel p_y was varied depending on flange thicknesses.

2.2.2.2 Plastic design of semi-continuous braced frames

For ultimate limit states, plastic design provides as attractive approach, particularly if linked with partial-strength joints providing a moment resistance 30-50% of that of the connected beam. As will be shown, such resistance still provides worthwhile reductions in beam weight or depth, and overall frame cost. The advantages of this approach are:

1. The moment resistance required at the connection is readily determined from a beam-type plastic hinge mechanism (Fig.2.4).

2. Ductility can be provided through use of relatively thin end plates (12-15mm) in mild steel, in conjunction with appropriately strong bolts and welds[2.9].
3. With such end plates, joint resistance will usually be independent of column size, thereby assisting the preparation of concise design tables for standard connections and permitting beam design to be completed before the columns are considered.
4. Stiffness calculations, which would necessarily include contributions from the column section, are avoided.
5. For typical relative values of dead and live loading, pattern loading need not be considered because each joint will attain its design resistance M_{Rd} under the factored dead load (Fig 2.5).
6. Although the beam's compression flange is unrestrained near the supports, the limited joint's resistance will reduce the likelihood that lateral buckling will occur.

2.2.3 Design procedures in semi-continuous construction.

2.2.3.1 Design of the beams.

Two aspects of the "simple" design procedure from BS 5950[2.7] encourage the use of semi-continuous design:

1. In "simple" design, beam end reactions are assumed to act at an eccentricity of 100 mm from the face of the column (Figure 2.6), to account very approximately for the observed semi-rigid nature of nominally-pinned joints. So unbalanced beam loading causes bending moment in columns. This is even though the beams themselves are still designed as simply-supported, with the effective span taken as the distance between

centres of columns. The eccentricity moment to some extent offsets the moments induced in columns with semi-continuous construction.

2. The deflection limits in BS 5950[2.7] apply under imposed load only. Consequently, serviceability calculations for semi-continuous design can be simplified, often by retaining the assumption of pinned joints, without sacrificing economy except for some frames designed with S355 steel. Calculations of joint stiffness will not normally be necessary.

In semi-continuous construction members were designed for a local plastic hinge mechanism, taking into account the design moment resistance of the joints. Beams were assumed to be laterally restrained by the floor or roof units. Unlike conventional simple design, the beam was taken to span between the flanges of the columns, assuming the column sections obtained in simple design. This was because accurate account was being taken of the moment developed in the partial-strength connection at the face of the column. The total load on the beam was not reduced though in comparison with simple design. The end moments were selected from tables originally provided in reference[2.6] for wind-moment joints, because it is these configurations that have the assured ductility. Two strategies were used to select beam sizes, based on (i) minimum depth and (ii) minimum weight. The beam section selected had to be at least "compact"[2.7] to enable its plastic moment to be developed; a restriction to only "plastic" sections was unnecessary as the plastic hinge in the beam section is always the last to form due to the limited resistance of the connections. Beam sizes were selected from the list of Universal Beams to provide adequate resistance and stiffness. The deflections of the beams were calculated as for a "simple" beam for the reasons already explained; the limit was checked according to BS 5950 Part 1[2.7]. This assumption did not affect beam sizes, except with some of the frames designed with S355 steel.

2.2.3.2 Design of the columns

For design of the columns the effective length factor about the minor axis was taken as 0.85, as for simple design. The moment applied to a column was taken as the moment resistance of the connection plus the additional eccentric moment arising from the presence of the joint at the face of the column. The latter moment was therefore determined using an eccentricity of half the depth of the column section. The external columns thereby carried axial load and end moment whereas the internal columns in the studies carried only axial load.

The buckling resistance moment for the column section was calculated in accordance with the formula given in Appendix B of BS 5950 Part 1[2.7]. The formula for slenderness was taken to be :-

$\lambda_{LT} = nuv\lambda$

where $n = 1$,

u = the buckling parameter,

v = the slenderness factor.

The compressive strength, p_c was calculated in accordance with the formula given in Appendix C of BS 5950.

For partial strength connections, columns were checked against overall buckling using the simplified approach outlined in BS 5950: Part 1 clause 4.8.3.3.1 with moment factor taken to be 0.43. Bending moment diagrams are assumed to form at least partial double curvature on the column as shown in Fig. 2.7. The beam end moment M_{beam} is assumed to be divided equally between the upper and lower column lengths. A further check on the local capacity was made using equations in BS 5950: Part 1 clause 4.8.3.2. The relevent equations are given in 1.7.3.1 above, but with M_y taken as zero. All column members were Universal Columns.

2.3 Partial strength connections

As previously mentioned, beams were designed for a local plastic hinge mechanism taking into account of the moment resistance in connections, with ductility assured by testing[2.9]. For the partial-strength connections, failure of the end-plate, or the column flanges to which it is attached, can be modelled as an equivalent T-stub flanges as illustrated in Eurocode 3: Part 1:1 and shown in Figure 2.8. The resistance of a beam-to-column connection may also depend on the strength of the beam's flanges, the bolts in the connections, the welds between the beam and end plate, and the resistance of the column web. There are three possible modes of failure for the end plate and the column flange:

Mode 1 Yielding of column flange and/or end plate only.

Mode 2 Combination of yielding of column flange and/or end plate with bolt failure.

Mode 3 Bolt failure only.

To ensure sufficient ductility, strictly only Mode 1 or, with calculation, Mode 2 failure is permitted[2.1], leading generally to the use of thin end plates. The use of thin end plates also ensures that usually it is the resistance of this component that governs the resistance of the entire connection, provided that Grade 275 steel is used in conjunction with M20 grade 8.8 bolts and suitably robust welds. This permits the moment resistance of standardised connections to be tabulated in a form which is dependent only on the depth of the beam. This greatly eases the task of design. Two types of partial strength connections namely flush end plate (W1/20) and extended end plate (W4/20) connections were used in this study. The latter is expected to reduce beam sections more than the former. The standard tables[2.10] used in the study are shown in Table 2.1 and Table 2.2 respectively.

The tables for moment connections include the following checks on the local resistance of the column: web tension, web crushing, web buckling, web shear, and flange bending. Excepting web shear though, none of these elements is critical, provided column sections are at least 203x203x52UC for flush end plate connection (Fig.2.3(a)). Checks on the 152x152x37UC section are not included in the standard table[2.6]; therefore the force resistance of 180kN was calculated from the methods explained in the Steel Construction Institute publication[2.6]. For the 203x203x46UC section, the force resistance of the connection is given in the standard table[2.6] as 198kN, which can be used to calculate the moment resistance for each beam size. Therefore, the moment resistance for both 152x152x37UC and 203x203x46UC sections associated with different beam sizes were calculated and are listed in Table 2.3. The following formulae were used to calculate the moment resistance of the connection listed in Table 2.3:

M.R (in. N.m) = $180(h-0.5t_f-60)$ for 152x152x37UC

M.R (in. N.m) = $198(h-0.5t_f-60)$ for 203x203x46UC

where,

M.R is the moment resistance of the connection depending on the size of beam,

h is the depth of the beam, and

t_f is the thickness of the beam.

For extended end plate connection (Fig 2.3(b)), none of the column elements checked for local resistance are critical for column section greater than or equal to 203x203x71UC. For moment resistance of the connection with beam connected to column sections less than 203x203x71UC, the following formulae[2.6] were used:

M.R (in. N.m) = $124(h-0.5t_f+40)+107(h-0.5t_f-60)$ for 203x203x46UC

M.R (in. N.m) = $124(h-0.5t_f+40)+181(h-0.5t_f-60)$ for 203x203x52UC

M.R (in. N.m) = $124(h-0.5t_f+40)+191(h-0.5t_f-60)$ for 203x203x60UC

For frames designed with S355 steel, the tension bolt forces of the connections for critical columns are given in standard tables[2.6].

2.4 Approach used to calculate total weight

The total weight calculated for both simple and semi-continuous construction takes into account all beams, columns, and fittings. The fittings include the end plates and the column base plates. For column base plates, standard tables are available[2.5]. The beam's weight was calculated as mass of beam per metre multiplied by the clear span; the latter is defined as the length between the faces of the column supports. The calculations for column weight take account of the effective lengths given in Table 2.4 where H is the height of the storey taken from base plate or centre of beam to centre of beam, D is the depth of beam and US is the upstand of the column above the roof beam needed to develop the connections resistance in the semi-continuous schemes, taken as 25mm for flush end plate joints and 90mm for extended end plates. The dimension of 500mm relates to the position of column splices.

Table 2.4 is illustrated in Figure 2.9 for an external column in simple construction and Figure 2.10 for semi-continuous construction. Typical calculations of the total weight using a spreadsheet[2.15] are given in tabular form in Table 2.5 and Table 2.6 for a two-storey two-bay frame, designed with beams spanning 9 metre between column centres and connected by flush end plate connections. Table 2.5(b) shows that the length of the second-storey does not change even though an upstand has to be included in semi-continuous design; this is because the depth of the designed beam has been reduced compared with the simple construction (Table 2.5(a)). The number of columns shown in

Table 2.6 relates to the number of inter-storey lengths. The details provided to enable weight and prices to be calculated are given in Fig. 2.11 to 2.20.

2.5 Discussions and analysis of results.

The results of the percentage weight savings and an additional increase in percentage savings due to effect of increasing the number of bays from two to four are shown in Table 2.7(a-d) for a plane frame designed for S275 steel and in Table 2.8(a-d) for design in S355 steel. The frame arrangements studied and the dimensions and loading, are listed in Table 2.9 to Table 2.14 for S275 steel and Table 2.15 to Table 2.20 for S355 steel. Table 2.9 and Table 2.15 concern simple construction design with beams designed for minimum weight and Table 2.10 and Table 2.16 concern simple construction design with beams designed for minimum depth. The rest of the tables are concerned with frames designed for semi-continuous construction. A slight decrease in the design moment was due to reduction in an effective beam length as the column increase in size. A slight change in the moment capacity of a connection at an external column may be observed; this was due to a decrease in the bolt force for columns local resistance is critical. Only moment resistance for external columns were listed in the tables because most of the critical columns happened to be external. However, the difference between moment resistance for external and internal columns is not that significant. The connection moments listed were taken from Table 2.1-2.3 and formulae given in Section 2.3 for critical columns. The scope for savings was studied with regard to both cost and weight. The analysis for the former is discussed and analysed in Section 2.5.1 below.

In comparing the two forms of construction, the moment resistance of the flush end plate and extended end plate connections meant that beams with partial-strength connections were of smaller depth and lighter section. Although moment is transferred to the external column due to beam end moments, in this study there was no increase in weight of external columns except for some frames designed with extended end plates. This is because the use of extended end plate contributes to a higher moment resistance which transfers to the external column. In these cases, if the beam is designed for minimum weight, this is more likely to increase the column size, compared with a beam designed for minimum depth, because the deeper section results in a higher moment resistance at the joint. However, the use of an equivalent uniform moment factor of 0.43 in stability checks reduced some external columns compared to simple design. This is shown in Frame FEPMW7 (Table 2.11), in Frame FEPMD12 (Table 2.17) and Frame FEPMW12 (Table Table 2.19).

The use of a 152x152x37UC as an external column results in a smaller moment resistance for the joint than at the internal column of 203x203x46UC as shown in Fig. 2.21. However, this difference in moment resistance did not change the selected beam section from that needed for an internal bay.

Within the scope of the study, the percentage savings depend on the number of storeys, the beam span of each frame, the compactness of available sections, and the depth and deflection limit of the beam selected. The overall percentage of weight savings in steel ranging between 2.38% to 11.95% for S275 steel and -0.56% to 8.95% for S355 steel. The variations in the range of percentage of savings are due to the following main reasons:-

- In Frames FEPMD9, FEPMD10 (Table 2.17), EEPMD9 and EEPMD10 (Table 2.18), the floor beam is selected as a 356x171x51UB, instead of the 356x171x45UB which has adequate stiffness and also, at first sight, adequate resistance. However, the latter section is "semi-compact" (Class 3) in S355 steel and cannot therefore by permitted to be designed plastically. The next beam section to be selected is 356x127x39UB. However, due to deflection limitation, this section is not been selected. Hence, the heavier section is selected.
- In Frames EEPMW9 and EEPMW10 (Table 2.20), no further reduction in the beam weight can be achieved compared to design with flush end plates. This is because the next lightest sections are of less depth; the corresponding reduction of moment resistance in the joints then even though the joints are now of the extended type.
- For floor beam designed as 457x191x89UB in Frame FEPMD11 and FEPMD12 in Table 2.17 and in Frame EEPMD11 and EEPMD12 in Table 2.18, no further reduction in the beam weight can be achieved because of deflection limitations. The calculated deflection corresponding to the next lightest section (457x191x82UB) is 27.0 mm while the deflection limit[2.7] is L/360 = 25 mm.

2.5.1 Percentage of cost savings

Cost savings were determined for frames with flush end plate connections designed for beams of minimum depth using S275 steel. Costing was done by a fabricator in the United Kingdom dated September 1994. The costing was based on all materials designed for S275 steel and blast cleaned before fabrication. Example of costing for two bay two storey frame spanning at 6m for both simple construction and semi-continuous construction is shown in Table 2.21(a and b). The rate of costing was given in term of price in pounds per 1000 Kg. A slight increase in the costing rate for the same beam depth in partial strength joints was due to the need of more work on welding. An overall cost saving for the fabricated plane frame delivered and erected within 100 miles radius was about 5.5% against "simple" construction. Further cost savings on the frame are due to a reduction in the depth of floor construction[2.4], which leads to economy in cost of cladding, or an increase in usable height. A summary of the comparison of percentage weight and cost savings plotted against beam span, for frames beam designed for minimum beam depth is shown in Figure 2.22. The result shows that the average percentage of weight savings is about 7.5% which is 2% higher than cost savings.

2.5.2 Percentage of weight savings

Percentage weight savings were determined by dividing the total mass difference with the total mass of frame designed for simple construction. The total mass for each frame was calculated by including the mass of the beam section, column section, end plates (fittings), and column base plate. The mass of beam and column sections were calculated by multiplying the effective length with the number of designed sections. For end plate and column base plate, volume of the number of components were calculated and multiplied by the density of steel of 7850kg/m^3. Further results cover the percentage weight saving obtained with other design strategies:

- Effect of increasing the number of bays from two to four.
- Effect of using S355 steel.
- Effect of increasing the beam span from 6m to 9m.
- Effect of the use of extended end plate connections.
- Effect of selecting beam sections according to minimum weight instead of minimum depth.

2.5.2.1 Effect of increasing the number of bays from two to four

The additional increases in percentage savings of weight and the average values are shown in Table 2.22. The average results are calculated from a comparison of four bay and two bay schemes given in Tables 2.7 and 2.8. The overall results show an increase in percentage weight savings for all frames except for Frame FEPMW7 in Table 2.11, FEPMD12 in Table 2.17, and Frame FEPMW12 in Table 2.19. All these frames are of six storeys. The external column section for the bottom size is reduced in size compared to simple design, due to the equivalent uniform moment factor of 0.43 used to check stability in the semi-continuous design. This benefit becomes less significant in the four-bay schemes due to the greater influence of the massess of the beam and the internal columns on total weight.

2.5.2.2 Effect of using S355 steel for the member sections.

The range of percentage savings of weight due to effect of using S355 steel and its average value are summarised as shown in Table 2.23. The results show that the use of S355 steel decreases the savings from semi-continuous design in some frames. This is because the moment capacity of the connections, which does not alter with grade of steel, has relatively less influence with higher grade beams. Smaller beams selected due to higher grade steel contribute to smaller lever arm and it is therefore; less easy to develop the same level of moment, compared to S275 schemes. Also, some of the beams designed for S355 steel were not reduced in size due to the need for compactness and stiffness to meet the deflection limitations.

2.5.2.3 Effect of increasing the beam span from 6m to 9m.

Table 2.24(a) shows the effect of increasing the beam span from 6m to 9m. The results show that the percentage of savings decreases as the beam span increases from 6 to 9 metres. The use of 9 metre span is expected to result in a deeper beam section that contributes to a higher moment capacity in the connection which in turn would decrease the maximum design moment of the beam. However, the decrease in maximum design moment of the beam is more on 6m spans than on 9m. This can be demonstrated by calculating the ratio of moment resistance of the connection to the "free" design moment in the beam (i.e for simple construction) as shown in Table 2.24(b). The calculations were done only for S275 steel because the design of beams in this grade is governed only by the plastic modulus. The determination of maximum design moment in the semi-continuous frame is calculated as $wL^2/8$ minus the moment resistance of the connection. Therefore, the higher the ratio the lesser the maximum design moment of the beam for semi-continuous construction, which results in a smaller section.

2.5.2.4 Effect of the use of extended end plate connections.

Table 2.25 shows the range percentage savings of weight and the average values resulting from the use of extended end plate connections. The overall results show an increase in the percentage savings for the extended end plate connection in schemes using S275 steel. However, for the member designed in S355 steel, the use of extended end plate connections does not change the size of the floor beam from the 356x171x51UB shown in Table 2.17 and Table 2.18. This is because of the requirement for compactness as described in Section 2.5.

The use of extended end plate connections contributes to a higher moment capacity for the connection. This decreases the maximum design moment of the beam which increases the savings in these members. However, the use of extended end plates causes an increase in weight of an external column in some frames due to the greater beam end moment. Generally, however, the reduction of total beam weight is more than the increase in the weight of the external columns. Although other forms of extended end plate joints are available[2.6] with higher moment capacity, these have not been included in the study because it is expected that the weight in external columns will increase further and the design of beams may then be controlled by the deflection limitation.

2.5.2.5 Effect of selecting beam sections according to minimum weight instead of minimum depth.

Table 2.26 show the range of percentage savings and the average values due to the effect of selecting beam sections according to minimum weight instead of minimum depth. The results show that frames designed for minimum weight have a higher percentage of savings in S275 steel. This is because the beam weight is minimised by selecting the beam section according to minimum weight. For frames designed with S355 steel however, some of the beams designed for minimum weight do not result in higher savings. For the 533x210x82UB section as shown in Table 2.19 and Table 2.20, the selected beam section is the same as for simple construction. Although the maximum design moment of the beam has been reduced by the moment capacity of the connection, the beam of section 533x210x82UB is still selected. The next available beam to reduce the weight is 457x191x74UB. However, this section's plastic modulus is insufficient.

2.6 Conclusions

The benefits of semi-continuous construction are difficult to quantify because they depend upon what practice is followed in "simple" construction, and on the range of available sections. Partial-depth end plates with only web welds provide a very economical form of connection for "simple" design. Even so, studies shows an average overall cost saving for a planar frame of 5.5% for frames FEPMD1 to FEPMD4 (Table 2.14). This was achieved using plastic design methods in conjunction with published resistance tables for standard connections. With experience, design calculations therefore take a little longer than those for "simple" design. The flush end plate connections used for the semi-continuous designs were of limited moment resistance, with the result that the same column sections could be used for the two design approaches. However, the use of extended end plate connection results in the increase of column section. Further conclusions are as follows:

1. The use of partial-strength connections result in shallower beams and worthwhile reductions in the cost of the structure.
2. Increases in the number of bays contribute to a better weight savings for most of the frames.
3. Most frames designed with S275 steel show more savings though than frames designed with S355 steel.
4. Increasing the length a beam's span contributes to a reduction in the weight savings compared to frames designed with S355 steel.
5. The use of extended end plate connections contribute to higher percentage weight savings than flush end plates in most of the frames for S275 steel.
6. Beams designed for minimum weight contribute to better percentage weight savings for most of frames than beams designed for minimum depth.
7. Designs with deeper beam sections contribute higher percentage savings for most of the frames.
8. Some of the beams designed for S355 steel are governed by compactness and the deflection limitations.

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Table 2.1 Moment resistance for flush end plate connections.

W1/20

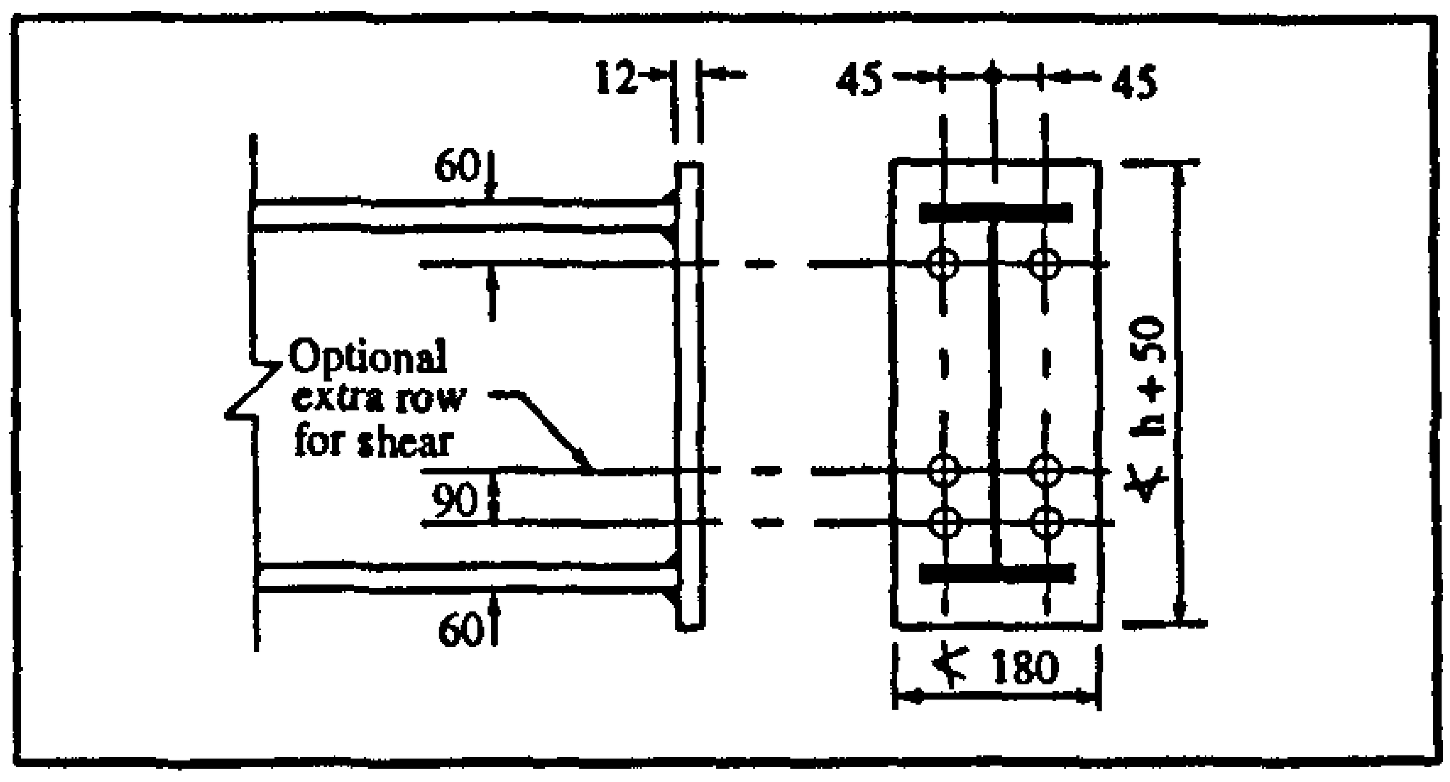
Box 1

Moment Resistance

M.R. (in Nm) = 208 (h - 0.5 t_f - 60)

t_f (mm)	h (mm)	Serial Size Mass per metre	Moment Resistance kNm
		762 x 267	
25.4	769.6	197 b	145
21.6	762.0	173 b	143
17.5	753.9	147	142
		686 x 254	
23.7	692.9	170 b	129
21.0	687.6	152 b	128
19.0	683.5	140 b	127
16.2	677.9	125	127
		610 x 229	
22.1	617.0	140 b	113
19.6	611.9	125 b	112
17.3	607.3	113	112
14.8	602.2	101	111
		533 x 210	
21.3	544.6	122 b	98
18.8	539.5	109 b	98
17.4	536.7	101	97
15.6	533.1	92	97
13.2	528.3	82	96
		457 x 191	
19.6	467.4	98 b	83
17.7	463.6	89 bw	82
16.0	460.2	82	81
14.5	457.2	74	81
12.7	453.6	67	80
		457 x 152	
18.9	465.1	82 b	82
17.0	461.3	74	82
15.0	457.2	67	81
13.3	454.7	60	81
10.9	449.8	52	80
		406 x 178	
16.0	412.8	74	72
14.3	409.4	67	71
12.8	406.4	60	71
10.9	402.6	54	70
		406 x 140	
11.2	402.3	46	70
8.6	397.3	39	69
		356 x 171	
15.7	364.0	67	61
13.0	358.6	57	61
11.5	355.6	51	60
9.7	352.0	45	60
		356 x 127	
10.7	352.8	39	60
8.5	348.5	33	59
		305 x 165	
13.7	310.9	54	51
11.8	307.1	46	50
10.2	303.8	40	50
		305 x 127	
14.0	310.4	48	51
12.1	306.6	42	50
10.7	303.8	37	49
		305 x 102	
10.8	312.7	33	51
8.9	308.9	28	51
6.8	304.8	25	50
		254 x 146	
12.7	259.6	43	40
10.9	256.0	37	40
8.6	251.5	31	39
		254 x 102	
10.0	260.4	28	41
8.4	257.0	25	40
6.8	254.0	22	40

b - Where t_f > 18 use a butt weld to the flange
bw- If the beam is S275 use a butt weld to the flange



Box 2

Column Limitations

$\Sigma F_b \not> 208$ kN

See Note 2

S275					Grade	S355				
v	iv	iii	ii	i	Zone	i	ii	iii	iv	v
Web Tension	Web Crushing	Web Buckling	Web Shear	Flange Bending	Serial Size Mass per metre	Flange Bending	Web Shear	Web Buckling	Web Crushing	Web Tension
					356 x 368					
✓	✓	✓	✓	✓	202	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	177	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	153	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	129	✓	✓	✓	✓	✓
					305 x 305					
✓	✓	✓	✓	✓	283	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	240	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	198	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	158	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	137	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	118	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	97	✓	✓	✓	✓	✓
					254 x 254					
✓	✓	✓	✓	✓	167	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	132	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	107	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	89	✓	✓	✓	✓	✓
✓	✓	✓	406	✓	73	✓	✓	✓	✓	✓
					203 x 203					
✓	✓	✓	✓	✓	86	✓	✓	✓	✓	✓
✓	✓	✓	395	✓	71	✓	✓	✓	✓	✓
✓	✓	✓	349	✓	60	✓	✓	✓	✓	✓
✓	✓	✓	300	✓	52	✓	387	✓	✓	✓
✓	✓	✓	273	■	46	✓	353	✓	✓	✓

■ - Less than ΣF_b

(shaded) - Sections are not designated class 1

Box 3

Shear Resistance

See Note 4

236 kN

(optional extra row adds 184 kN)

Table 2.2 Moment resistance for extended end plate connections **W4/20**

Box 1

Moment Resistance

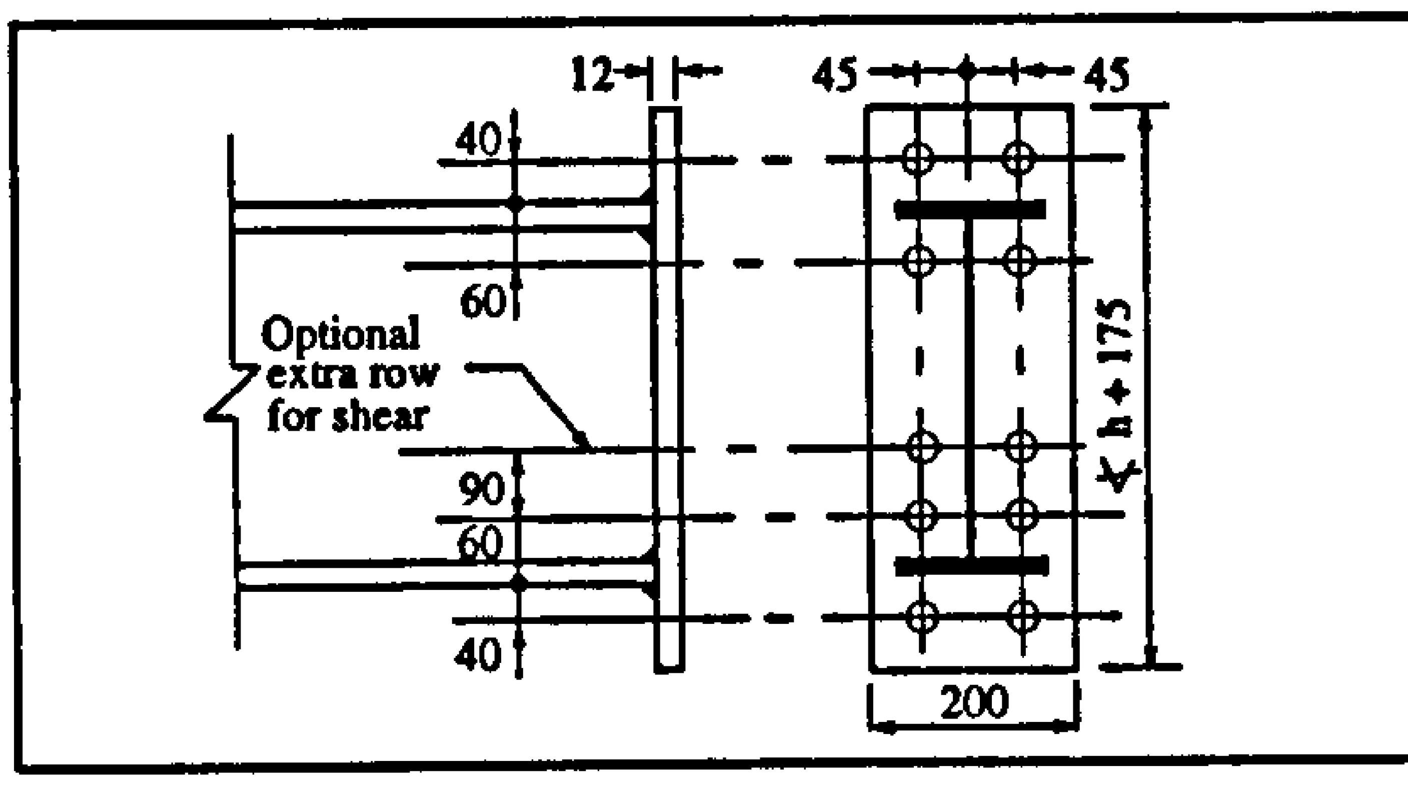
M.R. (in Nm) = 124 (h - 0.5 t_f + 40)

208 (h - 0.5 t_f - 60)

t_f (mm)	h (mm)	Serial Size Mass per metre	Moment Resistance kNm
		533 x 210	
21.3	544.6	122 b	169
18.8	539.5	109 b	168
17.4	536.7	101	167
15.6	533.1	92	167
13.2	528.3	82	165
		457 x 191	
19.6	467.4	98 b	144
17.7	463.6	89 bw	143
16.0	460.2	82	142
14.5	457.2	74	142
12.7	453.6	67	141
		457 x 152	
18.9	465.1	82 b	143
17.0	461.3	74	142
15.0	457.2	67	141
13.3	454.7	60	141
10.9	449.8	52	140
		406 x 178	
16.0	412.8	74	127
14.3	409.4	67	126
12.8	406.4	60	125
10.9	402.6	54	124
		406 x 140	
11.2	402.3	46	124
8.6	397.3	39	123
		356 x 171	
15.7	364.0	67	110
13.0	358.6	57	109
11.5	355.6	51	108
9.7	352.0	45	107
		356 x 127	
10.7	352.8	39	108
8.5	348.5	33	107
		305 x 165	
13.7	310.9	54	93
11.8	307.1	46	92
10.2	303.8	40	91
		305 x 127	
14.0	310.4	48	93
12.1	306.6	42	92
10.7	303.8	37	91
		305 x 102	
10.8	312.7	33	94
8.9	308.9	28	93
6.8	304.8	25	92
		254 x 146	
12.7	259.6	43	76
10.9	256.0	37	75
8.6	251.5	31	74
		254 x 102	
10.0	260.4	28	77
8.4	257.0	25	76
6.8	254.0	22	76

b - Where t_f > 18 use a butt weld to the flange

bw- If the beam is S275 use a butt weld to the flange



Box 2

Column Limitations

$\Sigma F_b \ngtr 338$ kN

See Note 2

S275					Grade	S355				
v	iv	iii	ii	i	Zone	i	ii	iii	iv	v
Web Tension	Web Crushing	Web Buckling	Web Shear	Flange Bending	Serial Size Mass per metre	Flange Bending	Web Shear	Web Buckling	Web Crushing	Web Tension
					356 x 368					
✓	✓	✓	✓	✓	202	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	177	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	153	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	129	✓	✓	✓	✓	✓
					305 x 305					
✓	✓	✓	✓	✓	283	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	240	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	198	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	158	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	137	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	118	✓	✓	✓	✓	✓
✓	✓	✓	565	✓	97	✓	✓	✓	✓	✓
					254 x 254					
✓	✓	✓	✓	✓	167	✓	✓	✓	✓	✓
✓	✓	✓	✓	✓	132	✓	✓	✓	✓	✓
✓	✓	✓	613	✓	107	✓	✓	✓	✓	✓
✓	✓	✓	497	✓	89	✓	642	✓	✓	✓
✓	✓	✓	406	✓	73	✓	525	✓	✓	✓
					203 x 203					
✓	✓	✓	497	✓	86	✓	642	✓	✓	✓
✓	✓	✓	395	✓	71	✓	510	✓	✓	✓
✓	✓	✓	349	✓	60	✓	451	✓	✓	✓
✓	✓	✓	300*	✓	52	✓	387	✓	✓	✓
✓	(■)	■	273*	■	46	✓	353	✓	✓	✓

* - Less than ΣF_b

(shaded) - Sections are not designated class 1

(■)- Re - calculate

Box 3

Shear Resistance

See Note 4

236 kN

(optional extra row adds 184 kN)

Table 2.3 : Moment resistance of connection for beam connected to 152x152x37UC and 203x203x46UC

Thickness (mm)	Depth (mm)	Serial Size	Mass per (m)	M.R for 203x203x46 UC	M.R for 152x152x37 UC
18.9	465.1	457 x 152	82	78	71
17.0	461.3		74	78	71
15.0	457.2		67	77	70
13.3	454.7		60	77	70
10.9	449.8		52	76	69
16.0	412.8	406 x 178	74	68	62
14.3	409.4		67	68	62
12.8	406.4		60	67	61
10.9	402.6		54	67	61
11.2	402.3	406 x 140	46	67	61
8.6	387.3		39	66	60
15.7	364.0	356 x 171	67	59	53
13.0	358.6		57	58	53
11.5	355.6		51	57	52
9.7	352.0		45	57	52
10.7	352.8	356 x 127	39	57	52
8.5	348.5		33	56	51
13.7	310.9	305 x 165	54	48	44
11.8	307.1		46	48	43
10.2	303.8		40	47	43
14.0	310.4	305 x 127	48	48	44
12.1	306.6		42	48	43
10.7	303.8		37	47	43
10.8	312.7	305 x 102	33	49	45
8.9	308.9		28	48	44
6.8	304.8		25	48	43
12.7	259.6	254 x 146	43	38	35
10.9	256.0		37	38	34
8.6	251.5		31	37	34
10.0	260.4	254 x 102	28	39	35
8.4	257.0		25	38	35
6.8	254.0		22	38	34

Table 2.4: Calculation of effective length for column weight in semi-continuous and simple construction.

	Columns	
Storey	Semi-continuous	Simple construction
4	H + D/2 mm + US	H + D/2
3	H - D/2 - 500 mm	H - D/2 - 500 mm
2	H + D/2 + 500 mm	H + D/2 + 500 mm
1	H only	H only

Table 2.5 (a) : Calculation of total mass of beams and columns for simple construction

	UB section	Clear span (m)	Total mass of beam (kg)	Position of column	UC section	Effective length (m)	Total mass of each column (kg)
2nd. Storey	457x191x82	8.79	1441.56	External Internal External	203x203x46 203x203x60 203x203x46	4.229	194.53 253.74 194.53
1st. Storey	533x210x109	8.79	1916.22	External Internal External	203x203x46 203x203x60 203x203x46	5.0	230.0 300.0 230.0
Total mass			3357.78				1402.80

Table 2.5 (b) : Calculation of total mass of beams and columns for semi-continuous construction with flush end plate connection.

	UB section	Clear span (m)	Total mass of beam (kg)	Position of column	UC section	Effective length (m)	Total mass of each column (kg)
2nd. Storey	406x178x74	8.79	1300.92	External Internal External	203x203x46 203x203x60 203x203x46	4.228	194.48 253.68 194.48
1st. Storey	533x210x92	8.79	1617.36	External Internal External	203x203x46 203x203x60 203x203x46	5.0	230.0 300.0 230.0
Total mass			2918.28				1402.64

Table 2.6(a) Calculation of total mass for simple construction for 2 bay, 2 storey, and 9m span frame.

			Weight in (kg) for number of component required		
Component	Section	Number	Shaft	Fittings	Total
Roof beam	457x191x82	2	1441.56	10.93	1452.5
Floor beam	533x210x109	2	1916.22	22.61	1938.8
External column	203x203x46	4	849.06	0	849.06
Internal column	203x203x60	2	553.74	0	553.74
Column base plate	400x400x20	3	0	75.36	75.36
Total					4869.5

Table 2.6(b) Calculation of total mass for semi-continuous construction for 2 bay, 2 storey, and 9m span frame.

			Weight in (kg) for number of component required		
Component	Section	Number	Shaft	Fittings	Total
Roof beam	406x178x74	2	1300.92	34.36	1335.3
Floor beam	533x210x92	2	1617.36	43.93	1661.3
External column	203x203x46	4	848.96	0	849.96
Internal column	203x203x60	2	553.68	0	553.68
Column base plate	400x400x20	3	0	75.36	75.36
Total					4475.6
Total mass different = 393.9 kg					
Percentage weight saving = 8.09%					

Table 2.7 Savings in weight using sections in S275 steel.

Table 2.7(a) Braced frames; S275 steel; flush end plate partial-strength joints; 6m span.

	Flush end plate (min. depth) Beam span 6 metre			Flush end plate (min. weight) Beam span 6 metre		
	2 bay	4 bay	Add. increase	2 bay	4 bay	Add. increase
2 storey	7.64%	8.34%	0.7%	10.03%	10.33%	0.30%
4 storey	6.91%	7.36%	0.45%	9.86%	10.07%	0.21%
6 storey	5.92%	6.35%	0.43%	8.76%	9.61%	0.85%
8 storey	5.63%	5.91%	0.24%	8.12%	9.00%	0.88%

Table 2.7(b) Braced frames; S275 steel; flush end plate partial-strength joints; 9m span..

	Flush end plate (min. depth) Beam span 9 metre			Flush end plate (min. weight) Beam span 9 metre		
	2 bay	4 bay	Add. increase	2 bay	4 bay	Add. increase
2 storey	8.09%	8.15%	0.06%	8.15%	8.43%	0.28%
4 storey	8.64%	9.13%	0.49%	6.27%	6.57%	0.30%
6 storey	8.23%	8.61%	0.38%	8.17%	6.87%	-1.30%
8 storey	7.89%	8.32%	0.43%	4.85%	5.08%	0.23%

Table 2.7(c) Braced frames; S275 steel; extended end plate partial-strength joints; 6m span.

	Extended end plate (min. depth) Beam span 6 metre			Extended end plate (min. weight) Beam span 6 metre		
	2 bay	4 bay	Add. increase	2 bay	4 bay	Add. increase
2 storey	8.98%	11.27%	2.29%	9.38%	10.33%	0.28%
4 storey	9.55%	11.55%	2.00%	9.67%	11.67%	0.30%
6 storey	10.84%	11.86%	1.02%	10.78%	11.95%	1.17%
8 storey	10.47%	11.59%	1.12%	6.76%	9.65%	0.23%

Table 2.7(d) Braced frames; S275 steel; extended end plate partial-strength joints;9m span.

	Extended end plate (min. depth) Beam span 9 metre			Extended end plate (min. weight) Beam span 9 metre		
	2 bay	4 bay	Add. increase	2 bay	4 bay	Add. increase
2 storey	2.38%	5.26%	2.88%	4.01%	9.25%	5.24%
4 storey	4.69%	6.00%	1.31%	8.40%	10.73%	2.33%
6 storey	4.79%	5.56%	0.77%	8.14%	9.82%	1.68%
8 storey	4.56%	5.26%	0.70%	8.39%	9.63%	1.24%

Table 2.8 Savings in weight using sections in S355 steel.

Table 2.8(a) Braced frames; S355 steel; flush end plate partial-strength joints; 6m span.

	Flush end plate (min. depth) Beam span 6 metre			Flush end plate (min. weight) Beam span 6 metre		
	2 bay	4 bay	Add. increase	2 bay	4 bay	Add. increase
2 storey	3.27%	3.98%	0.71%	7.32%	7.81%	0.49%
6 storey	3.22%	3.48%	0.26%	8.31%	8.95%	0.64%

Table 2.8(b) Braced frames; S355 steel; flush end plate partial-strength joints; 9m span.

	Flush end plate (min. depth) Beam span 9 metre			Flush end plate (min. weight) Beam span 9 metre		
	2 bay	4 bay	Add. increase	2 bay	4 bay	Add. increase
2 storey	5.34%	5.63%	0.29%	1.53%	1.86%	0.33%
6 storey	7.26%	6.41%	-0.85%	2.94%	1.14%	-1.80%

Table 2.8(c)Braced frames; S355 steel; extended end plate partial-strength joints; 6m span.

	Extended end plate (min. depth) Beam span 6 metre			Extended end plate (min. weight) Beam span 6 metre		
	2 bay	4 bay	Add. increase	2 bay	4 bay	Add. increase
2 storey	2.48%	2.79%	0.31%	6.23%	6.93%	0.70%
6 storey	2.44%	3.13%	0.69%	7.52%	8.20%	0.68%

Table 2.8(d) Braced frames; S355 steel; extended end plate partial-strength joints; 9m span.

	Extended end plate (min. depth) Beam span 9 metre			Extended end plate (min. weight) Beam span 9 metre		
	2 bay	4 bay	Add. increase	2 bay	4 bay	Add. increase
2 storey	4.69%	4.98%	0.29%	1.78%	3.43%	1.65%
6 storey	4.31%	4.52%	0.21%	-0.56%	-0.08%	0.48%

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Gravity Load Floor LL	Gravity Load Roof DL	Gravity Load Roof LL	Cladding + Self Weight (kN) Floor (DL) Ext. Col	Floor (DL) Int Col	Roof (DL) Ext. Col	Roof (DL) Int. Col	Simple construction design (Flexible end plate) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Required design values Bending Moment (kNm) Floor	Bending Moment Roof	Shear Force (kN) Floor	Shear Force Roof
2 Storey 2 and 4 Bays	SCMW1	6.0 (Precast floor)													1st 457x152x60	406x140x46	Upto 2nd Storey	203x203x46	203x203x46	1st 324	216	216	144
4 Storeys 2 and 4 Bays	SCMW2														1st 457x152x60	406x140x46	Upto 2nd Storey	203x203x60	203x203x71	1st 324	216	216	144
															2nd 457x152x60					2nd 324		216	
															3rd 457x152x60		2nd-4th Storey	152x152x37	203x203x46	3rd 324		216	
6 Storeys 2 and 4 Bays	SCMW3														1st 457x152x60	406x140x46	Upto 3rd Storey	203x230x86	254x254x89	1st 324	216	216	144
															2nd 457x152x60					2nd 324		216	
															3rd 457x152x60					3rd 324		216	
															4th 457x152x60		3rd-6th Storey	203x203x46	203x203x46	4th 324		216	
															5th 457x152x60					5th 324		216	
8 Storeys 2 and 4 Bays	SCMW4														1st 457x152x60	406x140x46	Upto 3rd Storey	254x254x89	254x254x132	1st 324	216	216	144
															2nd 457x152x60					2nd 324		216	
															3rd 457x152x60					3rd 324		216	
															4th 457x152x60		3rd-6th Storey	203x203x60	203x203x71	4th 324		216	
															5th 457x152x60					5th 324		216	
															6th 457x152x60		6th-8th Storey	152x152x37	203x203x46	6th 324		216	
															7th 457x152x60					7th 324		216	
			5.0	4.0	2	6.0	4.0	4.0	4.0	1.5	55	7	30	7									
2 Storey 2 and 4 Bays	SCMW5	9.0 (Precast floor)													1st 610x229x101	533x210x82	Upto 2nd Storey	203x203x46	203x203x60	1st 729	486	324	216
4 Storeys 2 and 4 Bays	SCMW6														1st 610x229x101	533x210x82	Upto 2nd Storey	203x203x86	254x254x89	1st 729	486	324	216
															2nd 610x229x101					2nd 729		324	
															3rd 610x229x101		2nd-4th Storey	203x203x46	203x203x46	3rd 729		324	
6 Storeys 2 and 4 Bays	SCMW7														1st 610x229x101	533x210x82	Upto 3rd Storey	254x254x107	254x254x132	1st 729	486	324	216
															2nd 610x229x101					2nd 729		324	
															3rd 610x229x101					3rd 729		324	
															4th 610x229x101		3rd-6th Storey	203x203x52	203x203x71	4th 729		324	
															5th 610x229x101					5th 729		324	
8 Storeys 2 and 4 Bays	SCMW8														1st 610x229x101	533x210x82	Upto 3rd Storey	254x254x132	254x254x167	1st 729	486	324	216
															2nd 610x229x101					2nd 729		324	
															3rd 610x229x101					3rd 729		324	
															4th 610x229x101		3rd-6th Storey	203x203x86	254x254x107	4th 729		324	
															5th 610x229x101					5th 729		324	
															6th 610x229x101		6th-8th Storey	203x203x46	203x203x46	6th 729		324	
															7th 610x229x101					7th 729		324	

Table 2.9 Simple construction design using flexible end plate connections with beams designed for minimum weight (S275)

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Gravity Load Floor LL	Gravity Load Roof DL	Gravity Load Roof LL	Cladding + Self Weight (kN) Floor (DL) Ext. Col	Floor (DL) Int Col	Roof (DL) Ext. Col	Roof (DL) Int. Col	Simple construction design (Flexible end plate) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Required design values Bending Moment (kNm) Floor	Bending Moment Roof	Shear Force (kN) Floor	Shear Force Roof
2 Storey 2 and 4 Bays	SCMD1														1st 356x171x67	305x165x54	Upto 2nd Storey	203x203x46	203x203x46	1st 324	216	216	144
4 Storeys 2 and 4 Bays	SCMD2														1st 356x171x67 2nd 356x171x67 3rd 356x171x67	305x165x54	Upto 2nd Storey 2nd-4th Storey	203x203x60 152x152x37	203x203x71 203x203x46	1st 324 2nd 324 3rd 324	216	216 216 216	144
6 Storeys 2 and 4 Bays	SCMD3	6.0 (Precast floor)													1st 356x171x67 2nd 356x171x67 3rd 356x171x67 4th 356x171x67 5th 356x171x67	305x165x54	Upto 3rd Storey 3rd-6th Storey	203x230x86 203x203x46	254x254x89 203x203x46	1st 324 2nd 324 3rd 324 4th 324 5th 324	216	216 216 216 216 216	144
8 Storeys 2 and 4 Bays	SCMD4														1st 356x171x67 2nd 356x171x67 3rd 356x171x67 4th 356x171x67 5th 356x171x67 6th 356x171x67 7th 356x171x67	305x165x54	Upto 3rd Storey 3rd-6th Storey 6th-8th Storey	254x254x89 203x203x60 152x152x37	254x254x132 203x203x71 203x203x46	1st 324 2nd 324 3rd 324 4th 324 5th 324 6th 324 7th 324	216	216 216 216 216 216 216 216	144
			5.0	4.0	2	6.0	4.0	4.0	4.0	1.5	55	7	30	7									
2 Storey 2 and 4 Bays	SCMD5														1st 533x210x109	457x191x82	Upto 2nd Storey	203x203x46	203x203x60	1st 729	486	324	216
4 Storeys 2 and 4 Bays	SCMD6														1st 533x210x109 2nd 533x210x109 3rd 533x210x109	457x191x82	Upto 2nd Storey 2nd-4th Storey	203x203x86 203x203x46	254x254x89 203x203x46	1st 729 2nd 729 3rd 729	486	324 324 324	216
6 Storeys 2 and 4 Bays	SCMD7	9.0 (Precast floor)													1st 533x210x109 2nd 533x210x109 3rd 533x210x109 4th 533x210x109 5th 533x210x109	457x191x82	Upto 3rd Storey 3rd-6th Storey	254x254x107 203x203x52	254x254x132 203x203x71	1st 729 2nd 729 3rd 729 4th 729 5th 729	486	324 324 324 324 324	216
8 Storeys 2 and 4 Bays	SCMD8														1st 533x210x109 2nd 533x210x109 3rd 533x210x109 4th 533x210x109 5th 533x210x109 6th 533x210x109 7th 533x210x109	457x191x82	Upto 3rd Storey 3rd-6th Storey 6th-8th Storey	254x254x132 203x203x86 203x203x46	254x254x167 254x254x107 203x203x46	1st 729 2nd 729 3rd 729 4th 729 5th 729 6th 729 7th 729	486	324 324 324 324 324 324 324	216

Table 2.10 Simple construction design using flexible end plate connections with beams designed for minimum depth (S275)

Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m^2) Floor DL	Floor LL	Roof DL	Roof LL	Cladding + Self Weight (kN) Floor (DL) Ext. Col	Floor (DL) Int Col	Roof (DL) Ext. Col	Roof (DL) Int. Col
5.0	4.0	2	6.0	4.0	4.0	4.0	1.5	55	7	30	7

Basic Frame Type	Frame Identification	Width of Bay (m)	Semi-continuous construction (Flush end plate) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Design moment for beam connected to ext. & int. column (kNm) Floor	Design moment Roof	Moment capacity of connection at external column (kN) Floor	Moment capacity Roof
2 Storey 2 and 4 Bays	FEPMW1	6.0 (Precast floor)	1st 406x140x46	356x127x33	Upto 2nd Storey	203x203x46	203x203x46	1st 237	143	67	56
4 Storeys 2 and 4 Bays	FEPMW2		1st 406x140x46	356x127x33	Upto 2nd Storey	203x203x60	203x203x71	1st 243	143	70	51
			2nd 406x140x46					2nd 243		70	
			3rd 406x140x46		2nd-4th Storey	152x152x37	203x203x46	3rd 242		61	
6 Storeys 2 and 4 Bays	FEPMW3		1st 406x140x46	356x127x33	Upto 3rd Storey	203x230x86	254x254x89	1st 241	143	70	56
			2nd 406x140x46					2nd 241		70	
			3rd 406x140x46					3rd 241		70	
			4th 406x140x46		3rd-6th Storey	203x203x46	203x203x46	4th 237		67	
			5th 406x140x46					5th 237		67	
8 Storeys 2 and 4 Bays	FEPMW4		1st 406x140x46	356x127x33	Upto 3rd Storey	254x254x89	254x254x132	1st 240	146	70	51
			2nd 406x140x46					2nd 240		70	
			3rd 406x140x46					3rd 240		70	
			4th 406x140x46		3rd-6th Storey	203x203x60	203x203x71	4th 243		70	
			5th 406x140x46					5th 243		70	
			6th 406x140x46		6th-8th Storey	152x152x37	203x203x46	6th 243		70	
			7th 406x140x46					7th 242		61	
2 Storey 2 and 4 Bays	FEPMW5	9.0 (Precast floor)	1st 533x210x109	457x191x82	Upto 2nd Storey	203x203x46	203x203x60	1st 622	398	93	77
4 Storeys 2 and 4 Bays	FEPMW6		1st 533x210x109	457x191x82	Upto 2nd Storey	203x203x86	254x254x89	1st 613	398	98	77
			2nd 533x210x109					2nd 613		98	
			3rd 533x210x109		2nd-4th Storey	203x203x46	203x203x46	3rd 622		98	
6 Storeys 2 and 4 Bays	FEPMW7		1st 533x210x109	457x191x82	Upto 3rd Storey	254x254x89	254x254x132	1st 610	395	98	77
			2nd 533x210x109					2nd 610		98	
			3rd 533x210x109					3rd 610		98	
			4th 533x210x109		3rd-6th Storey	203x203x52	203x203x71	4th 615		98	
			5th 533x210x109					5th 615		98	
8 Storeys 2 and 4 Bays	FEPMW8		1st 533x210x109	457x191x82	Upto 3rd Storey	254x254x132	254x254x167	1st 609	398	98	77
			2nd 533x210x109					2nd 609		98	
			3rd 533x210x109					3rd 609		98	
			4th 533x210x109		3rd-6th Storey	203x203x86	254x254x107	4th 612		98	
			5th 533x210x109					5th 612		98	
			6th 533x210x109		6th-8th Storey	203x203x46	203x203x46	6th 612		98	
			7th 533x210x109					7th 622		93	

Table 2.11 Semi-continuous construction design using flush end plate connections with beams designed for minimum weight (S275)

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Floor LL	Roof DL	Roof LL	Cladding + Self Weight (kN) Floor (DL) Ext. Col	Floor (DL) Int Col	Roof (DL) Ext. Col	Roof (DL) Int. Col	Semi-continuous construction (Extended end plate) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Design moment for beam connected to ext. & int. column (kNm) Floor	Roof	Moment capacity of connection at external column (kN) Floor	Roof
2 Storey 2 and 4 Bays	EEPMW1														1st 406x140x46	356x127x33	Upto 2nd Storey	203x203x46	203x203x46	1st 224	131	89	78
4 Storeys 2 and 4 Bays	EEPMW2														1st 406x140x39		Upto 2nd Storey	203x203x71	203x203x71	1st 190		123	
															2nd 406x140x39	356x127x33				2nd 190	131	123	78
															3rd 406x140x46		2nd-4th Storey	2203x203x46	203x203x46	3rd 225		89	
6 Storeys 2 and 4 Bays	EEPMW3														1st 406x140x39		Upto 3rd Storey	203x203x86	254x254x89	1st 188		123	
															2nd 406x140x39					2nd 188		123	
		6.0 (Precast floor)													3rd 406x140x39	356x127x33				3rd 188	131	123	78
															4th 406x140x46		3rd-6th Storey	203x203x46	203x203x46	4th 225		89	
															5th 406x140x46					5th 225		89	
8 Storeys 2 and 4 Bays	EEPMW4														1st 406x140x39		Upto 3rd Storey	254x254x107	254x254x132	1st 187		123	
															2nd 406x140x39					2nd 187		123	
															3rd 406x140x39					3rd 187		123	
															4th 406x140x39	356x127x33	3rd-6th Storey	203x203x60	203x203x71	4th 190	131	123	78
															5th 406x140x39					5th 190		117	
															6th 406x140x39		6th-8th Storey	203x203x46	203x203x46	6th 190		117	
															7th 406x140x46					7th 225		89	
			5.0	4.0	2	6.0	4.0	4.0	4.0	1.5	55	7	30	7									
2 Storey 2 and 4 Bays	EEPMW5														1st 533x210x82	457x152x60	Upto 2nd Storey	203x203x71	203x203x60	1st 547	334	165	141
4 Storeys 2 and 4 Bays	EEPMW6														1st 533x210x82		Upto 2nd Storey	203x203x86	254x254x89	1st 545		165	
															2nd 533x210x82	457x152x60				2nd 545	334	165	141
															3rd 533x210x82		2nd-4th Storey	203x203x71	203x203x46	3rd 547		165	
6 Storeys 2 and 4 Bays	EEPMW7														1st 533x210x82		Upto 3rd Storey	254x254x107	254x254x132	1st 542		165	
															2nd 533x210x82					2nd 542		165	
		9.0 (Precast floor)													3rd 533x210x82	457x152x60				3rd 542	334	165	141
															4th 533x210x82		3rd-6th Storey	203x203x71	203x203x71	4th 547		165	
															5th 533x210x82					5th 547		165	
8 Storeys 2 and 4 Bays	EEPMW8														1st 533x210x82		Upto 3rd Storey	254x254x132	254x254x167	1st 541		165	
															2nd 533x210x82					2nd 541		165	
															3rd 533x210x82					3rd 541		165	
															4th 533x210x82	457x152x60	3rd-6th Storey	203x203x86	254x254x107	4th 544	334	165	141
															5th 533x210x82					5th 544		165	
															6th 533x210x82		6th-8th Storey	203x203x71	203x203x46	6th 544		165	
															7th 533x210x82					7th 547		165	

Table 2.12 Semi-continuous construction design using extended end plate connections with beams designed for minimum weight (S275)

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Gravity Load Floor LL	Gravity Load Roof DL	Gravity Load Roof LL	Cladding + Self Weight (kN) Floor (DL) Ext. Col	Floor (DL) Int Col	Roof (DL) Ext. Col	Roof (DL) Int. Col	Semi-continuous construction (Extended end plate) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Design moment for beam connected to ext. & int. column (kNm) Floor	Design moment Roof	Moment capacity of connection at external column (kN) Floor	Moment capacity Roof
2 Storey 2 and 4 Bays	EEPMD1	6.0 (Precast floor)													1st 356x171x51	254x146x43	Upto 2nd Storey	203x203x46	203x203x46	1st 234	156	79	53
4 Storeys 2 and 4 Bays	EEPMD2														1st 365x171x45 2nd 356x171x45 3rd 356x171x51	254x146x43	Upto 2nd Storey 2nd-4th Storey	203x203x60 2203x203x46	203x203x71 203x203x46	1st 208 2nd 208 3rd 234	156	104 104 79	53
6 Storeys 2 and 4 Bays	EEPMD3														1st 365x171x45 2nd 365x171x45 3rd 365x171x45 4th 356x171x51 5th 356x171x51	254x146x43	Upto 3rd Storey 3rd-6th Storey	203x230x86 203x203x46	254x254x89 203x203x46	1st 204 2nd 204 3rd 204 4th 234 5th 234	156	107 107 107 79 79	53
8 Storeys 2 and 4 Bays	EEPMD4														1st 365x171x45 2nd 365x171x45 3rd 365x171x45 4th 365x171x45 5th 365x171x45 6th 365x171x45 7th 356x171x51	254x146x43	Upto 3rd Storey 3rd-6th Storey 6th-8th Storey	254x254x89 203x203x60 203x203x46	254x254x132 203x203x71 203x203x46	1st 204 2nd 204 3rd 204 4th 208 5th 208 6th 208 7th 234	156	107 107 107 104 104 104 79	53
			5.0	4.0	2	6.0	4.0	4.0	4.0	1.5	55	7	30	7									
2 Storey 2 and 4 Bays	EEPMD5	9.0 (Precast floor)													1st 457x191x98	406x178x67	Upto 2nd Storey	203x203x60	203x203x60	1st 576	356	135	120
4 Storeys 2 and 4 Bays	EEPMD6														1st 457x191x98 2nd 457x191x98 3rd 457x191x98	406x178x67	Upto 2nd Storey 2nd-4th Storey	203x203x86 203x203x60	254x254x89 203x203x46	1st 565 2nd 565 3rd 576	356	144 144 135	120
6 Storeys 2 and 4 Bays	EEPMD7														1st 457x191x98 2nd 457x191x98 3rd 457x191x98 4th 457x191x98 5th 457x191x98	406x178x67	Upto 3rd Storey 3rd-6th Storey	254x254x107 203x203x60	254x254x132 203x203x71	1st 563 2nd 563 3rd 563 4th 576 5th 576	356	144 144 144 135 135	120
8 Storeys 2 and 4 Bays	EEPMD8														1st 457x191x98 2nd 457x191x98 3rd 457x191x98 4th 457x191x98 5th 457x191x98 6th 457x191x98 7th 457x191x98	406x178x67	Upto 3rd Storey 3rd-6th Storey 6th-8th Storey	254x254x132 203x203x86 203x203x60	254x254x167 254x254x107 203x203x46	1st 562 2nd 562 3rd 562 4th 565 5th 565 6th 565 7th 601	370	144 144 144 144 144 144 135	120

Table 2.13 Semi-continuous construction design using extended end plate connections with beams designed for minimum depth (S275)

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Floor LL	Roof DL	Roof LL	Cladding + Self Weight (kN) Floor (DL) Ext. Col	Floor (DL) Int Col	Roof (DL) Ext. Col	Roof (DL) Int. Col	Semi-continuous construction (Flush end plate) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Design moment for beam connected to ext. & int. column (kNm)	Floor	Roof	Moment capacity of connection at external column (kN) Floor	Roof
2 Storey 2 and 4 Bays	FEPMD1	6.0 (Precast floor)													1st 356x171x57	305x127x42	Upto 2nd Storey	203x203x46	203x203x46	1st	255	161	58	48
4 Storeys 2 and 4 Bays	FEPMD2														1st 356x171x57	305x127x42	Upto 2nd Storey	203x203x60	203x203x71	1st	253	164	61	43
															2nd 356x171x57					2nd	253		61	
															3rd 356x171x57		2nd-4th Storey	152x152x37	203x203x46	3rd	259		53	
6 Storeys 2 and 4 Bays	FEPMD3														1st 356x171x57	305x127x42	Upto 3rd Storey	203x230x86	254x254x89	1st	251	161	61	48
															2nd 356x171x57					2nd	251		61	
															3rd 356x171x57					3rd	251		61	
															4th 356x171x57		3rd-6th Storey	203x203x46	203x203x46	4th	255		58	
															5th 356x171x57					5th	255		58	
8 Storeys 2 and 4 Bays	FEPMD4														1st 356x171x57	305x127x42	Upto 3rd Storey	254x254x89	254x254x132	1st	250	164	61	43
															2nd 356x171x57					2nd	250		61	
															3rd 356x171x57					3rd	250		61	
															4th 356x171x57		3rd-6th Storey	203x203x60	203x203x71	4th	253		61	
															5th 356x171x57					5th	253		61	
															6th 356x171x57		6th-8th Storey	152x152x37	203x203x46	6th	253		61	
															7th 356x171x57					7th	259		53	
			5.0	4.0	2	6.0	4.0	4.0	4.0	1.5	55	7	30	7										
2 Storey 2 and 4 Bays	FEPMD5	9.0 (Precast floor)													1st 533x210x92	406x178x74	Upto 2nd Storey	203x203x46	203x203x60	1st	618	406	92	68
4 Storeys 2 and 4 Bays	FEPMD6														1st 533x210x92	406x178x74	Upto 2nd Storey	203x203x86	254x254x89	1st	615	407	97	68
															2nd 533x210x92					2nd	615		97	
															3rd 533x210x92		2nd-4th Storey	203x203x46	203x203x46	3rd	621		97	
6 Storeys 2 and 4 Bays	FEPMD7														1st 533x210x92	406x178x74	Upto 3rd Storey	254x254x107	254x254x132	1st	614	405	97	72
															2nd 533x210x92					2nd	614		97	
															3rd 533x210x92					3rd	614		97	
															4th 533x210x92		3rd-6th Storey	203x203x52	203x203x71	4th	616		97	
															5th 533x210x92					5th	616		97	
8 Storeys 2 and 4 Bays	FEPMD8														1st 533x210x92	406x178x74	Upto 3rd Storey	254x254x132	254x254x167	1st	614	407	97	68
															2nd 533x210x92					2nd	614		97	
															3rd 533x210x92					3rd	614		97	
															4th 533x210x92		3rd-6th Storey	203x203x86	254x254x107	4th	615		97	
															5th 533x210x92					5th	615		97	
															6th 533x210x92		6th-8th Storey	203x203x46	203x203x46	6th	615		97	
															7th 533x210x92					7th	621		92	

Table 2.14 Semi-continuous construction design using flush end plate connections with beams designed for minimum depth (S275)

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m^2) Floor DL	Floor LL	Roof DL	Roof LL	Cladding + Self Weight (kN) Floor (DL) Ext. Col	Floor (DL) Int Col	Roof (DL) Ext. Col	Roof (DL) Int. Col	Simple construction (Flexible end plate) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Required design values Bending Moment (kNm) Floor		Bending Moment (kNm) Roof	Shear Force (kN) Floor	Shear Force (kN) Roof
2 Storey 2 and 4 Bays	SCMW9	6.0 (Precast floor)													1st 457x152x52	406x140x39	Upto 2nd Storey	203x203x46	203x203x46	1st	324	216	216	144
6 Storeys 2 and 4 Bays	SCMW10														1st 457x152x52	406x140x39	Upto 3rd Storey	203x203x71	203x203x86	1st	324	216	216	144
															2nd 457x152x52					2nd	324			
															3rd 457x152x52					3rd	324			
															4th 457x152x52		3rd-6th Storey	203x203x46	203x203x46	4th	324			
															5th 457x152x52					5th	324			
			5.0	4.0	2	6.0	4.0	4.0	4.0	1.5	55	7	30	7										
2 Storey 2 and 4 Bays	SCMW11	9.0 (Precast floor)													1st 533x210x82	457x191x67	Upto 2nd Storey	203x203x46	203x203x46	1st	729	486	324	216
6 Storeys 2 and 4 Bays	SCMW12														1st 533x210x82	457x191x67	Upto 3rd Storey	254x254x89	254x254x107	1st	729	486	324	216
															2nd 533x210x82					2nd	729		324	
															3rd 533x210x82					3rd	729		324	
															4th 533x210x82		3rd-6th Storey	203x203x46	203x203x60	4th	729		324	
															5th 533x210x82					5th	729		324	

Table 2.15 Simple construction using flexible end plate connections with beams designed for minimum weight (S355)

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Floor LL	Roof DL	Roof LL	Cladding + Self Weight (kN) Floor (DL) Ext. Col	Floor (DL) Int Col	Roof (DL) Ext. Col	Roof (DL) Int. Col	Simple construction (Flexible end plate) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Required design values Bending Moment (kNm) Floor	Bending Moment (kNm) Roof	Shear Force (kN) Floor	Shear Force (kN) Roof
2 Storey 2 and 4 Bays	SCMD13	6.0 (Precast floor)													1st 457x171x57	305x127x42	Upto 2nd Storey	203x203x46	203x203x46	1st 324	216	216	144
6 Storeys 2 and 4 Bays	SCMD14														1st 457x171x57	305x127x42	Upto 3rd Storey	203x203x71	203x203x86	1st 324	216	216	144
															2nd 457x171x57					2nd 324			
															3rd 457x171x57		3rd-6th Storey	203x203x46	203x203x46	3rd 324			
															4th 457x171x57					4th 324			
															5th 457x171x57					5th 324			
			5.0	4.0	2	6.0	4.0	4.0	4.0	1.5	55	7	30	7									
2 Storey 2 and 4 Bays	SCMD15	9.0 (Precast floor)													1st 457x191x98	406x178x74	Upto 2nd Storey	203x203x46	203x203x46	1st 729	486	324	216
6 Storeys 2 and 4 Bays	SCMD16														1st 457x191x98	406x178x74	Upto 3rd Storey	254x254x89	254x254x107	1st 729	486	324	216
															2nd 457x191x98					2nd 729		324	
															3rd 457x191x98		3rd-6th Storey	203x203x46	203x203x60	3rd 729		324	
															4th 457x191x98					4th 729		324	
															5th 457x191x98					5th 729		324	

Table 2.16 Simple construction using flexible end plate connections with beams designed for minimum depth(S355)

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column: Ground (m)	Height of Column: Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m^2): Floor DL	Floor LL	Roof DL	Roof LL	Cladding + Self Weight (kN): Floor (DL) Ext. Col	Floor (DL) Int Col	Roof (DL) Ext. Col	Roof (DL) Int. Col	Semi-continuous construction (Flush end plate): Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Designed moment for beam connected to ext. & int. column (kNm): Floor	Roof	Moment capacity of connection at external column (kN.m): Floor	Roof
2 Storey 2 and 4 Bays	FEPMD9	6.0 (Precast floor)													1st 356x171x51	254x146x37	Upto 2nd Storey	203x203x46	203x203x46	1st 253	169	60	40
6 Storeys 2 and 4 Bays	FEPMD10														1st 356x171x51	254x146x37	Upto 3rd Storey	203x203x71	203x203x86	1st 252	169	60	40
															2nd 356x171x51					2nd 252		60	
															3rd 356x171x51					3rd 252		60	
															4th 356x171x51		3rd-6th Storey	203x203x46	203x203x46	4th 253		60	
															5th 356x171x51					5th 253		60	
			5.0	4.0	2	6.0	4.0	4.0	4.0	1.5	55	7	30	7									
2 Storey 2 and 4 Bays	FEPMD11	9.0 (Precast floor)													1st 457x191x89	356x171x67	Upto 2nd Storey	203x203x46	203x203x46	1st 631	414	82	61
6 Storeys 2 and 4 Bays	FEPMD12														1st 457x191x89	356x171x67	Upto 3rd Storey	254x254x73	254x254x107	1st 626	414	82	61
															2nd 457x191x89					2nd 626		82	
															3rd 457x191x89					3rd 626		82	
															4th 457x191x89		3rd-6th Storey	203x203x46	203x203x60	4th 630		82	
															5th 457x191x89					5th 630		82	

Table 2.17 Semi-continuous construction using flush end plate connections with beams designed for minimum depth (S355)

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Floor LL	Roof DL	Roof LL	Cladding + Self Weight (kN) Floor (DL) Ext. Col	Floor (DL) Int Col	Roof (DL) Ext. Col	Roof (DL) Int. Col	Semi-continuous construction (Extended end plate) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Designed moment for beam connected to ext. & int. column (kNm) Floor	Roof	Moment capacity of connection at external column (kN.m) Floor	Roof
2 Storey 2 and 4 Bays	EEPMD9	6.0 (Precast floor)													1st 356x171x51	254x146x37	Upto 2nd Storey	203x203x46	203x203x46	1st 205	134	101	71
6 Storeys 2 and 4 Bays	EEPMD10														1st 356x171x51	254x146x37	Upto 3rd Storey	203x203x71	203x203x86	1st 204	134	108	71
															2nd 356x171x51					2nd 204		108	
															3rd 356x171x51					3rd 204		108	
															4th 356x171x51		3rd-6th Storey	203x203x46	203x203x46	4th 205		101	
															5th 356x171x51					5th 205		101	
			5.0	4.0	2	6.0	4.0	4.0	4.0	1.5	55	7	30	7									
2 Storey 2 and 4 Bays	EEPMD11	9.0 (Precast floor)													1st 457x191x89	356x171x67	Upto 2nd Storey	203x203x46	203x203x46	1st 570	365	133	103
6 Storeys 2 and 4 Bays	EEPMD12														1st 457x191x89	356x171x67	Upto 3rd Storey	254x254x89	254x254x107	1st 565	365	143	103
															2nd 457x191x89					2nd 565		143	
															3rd 457x191x89					3rd 565		143	
															4th 457x191x89		3rd-6th Storey	203x203x46	203x203x60	4th 569		133	
															5th 457x191x89					5th 569		133	

Table 2.18 Semi-continuous construction using extended end plate connections with beams designed for minimum depth (S355)

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Floor LL	Roof DL	Roof LL	Cladding + Self Weight (kN) Floor (DL) Ext. Col	Floor (DL) Int Col	Roof (DL) Ext. Col	Roof (DL) Int. Col	Semi-continuous construction (Flush end plate) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Designed moment for beam connected to ext. & int. column (kNm) Floor	Roof	Moment capacity of connection at external column (kN.m) Floor	Roof
2 Storey 2 and 4 Bays	FEPMW9	6.0 (Precast floor)													1st 406x140x39	356x127x33	Upto 2nd Storey	203x203x46	203x203x46	1st 244	150	69	59
6 Storeys 2 and 4 Bays	FEPMW10														1st 406x140x39	356x127x33	Upto 3rd Storey	203x203x71	203x203x86	1st 243	150	69	59
															2nd 406x140x39					2nd 243		69	
															3rd 406x140x39					3rd 243		69	
															4th 406x140x39		3rd-6th Storey	203x203x46	203x203x46	4th 244		69	
															5th 406x140x39					5th 244		69	
			5.0	4.0	2	6.0	4.0	4.0	4.0	1.5	55	7	30	7									
2 Storey 2 and 4 Bays	FEPMW11	9.0 (Precast floor)													1st 533x210x82	457x152x60	Upto 2nd Storey	203x203x52	203x203x46	1st 617	394	96	81
6 Storeys 2 and 4 Bays	FEPMW12														1st 533x210x82	457x152x60	Upto 3rd Storey	254x254x73	254x254x107	1st 612	394	96	81
															2nd 533x210x82					2nd 612		96	
															3rd 533x210x82					3rd 612		96	
															4th 533x210x82		3rd-6th Storey	203x203x46	203x203x60	4th 616		96	
															5th 533x210x82					5th 616		96	

Table 2.19 Semi-continuous construction using flush end plate connections with beams designed for minimum weight (S355)

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Floor LL	Roof DL	Roof LL	Cladding + Self Weight (kN) Floor (DL) Ext. Col	Floor (DL) Int Col	Roof (DL) Ext. Col	Roof (DL) Int. Col	Semi-continuous construction (Extended end plate) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Designed moment for beam connected to ext. & int. column (kNm) Floor	Roof	Moment capacity of connection at external column (kN.m) Floor	Roof
			5.0	4.0	2	6.0	4.0	4.0	4.0	1.5	55	7	30	7									
2 Storey 2 and 4 Bays	EEPMW9	6.0 (Precast floor)													1st 406x140x39	356x127x33	Upto 2nd Storey	203x203x46	203x203x46	1st 198	102	114	99
6 Storeys 2 and 4 Bays	EEPMW10	6.0 (Precast floor)													1st 406x140x39	356x127x33	Upto 3rd Storey	203x203x71	203x203x86	1st 189	102	123	99
															2nd 406x140x39					2nd 189		123	
															3rd 406x140x39					3rd 189		123	
															4th 406x140x39		3rd-6th Storey	203x203x46	203x203x46	4th 198		114	
															5th 406x140x39					5th 198		114	
2 Storey 2 and 4 Bays	EEPMW11	9.0 (Precast floor)													1st 533x210x82	457x152x52	Upto 2nd Storey	203x203x52	203x203x46	1st 548	335	157	133
6 Storeys 2 and 4 Bays	EEPMW12	9.0 (Precast floor)													1st 533x210x82	457x152x52	Upto 3rd Storey	254x254x89	254x254x107	1st 543	335	165	133
															2nd 533x210x82					2nd 543		165	
															3rd 533x210x82					3rd 543		165	
															4th 533x210x82		3rd-6th Storey	203x203x52	203x203x60	4th 547		157	
															5th 533x210x82					5th 547		157	

Table 2.20 Semi-continuous construction using extended end plate connections with beams designed for minimum weight (S355)

Table 2.21(a) Calculation of percentage in weight and cost savings for simple construction for 2 bay, 2 storey, and 6m span frame.

			Weight in (kg)			Costing in pounds	
Component	Section	Number	Shaft	Fittings	Total	Rate per 1000 Kg	Total
Roof beam	305x165x54	2	626.06	8.29	634.35	831	527.14
Floor beam	356x171x67	2	776.77	8.29	785.06	811	636.68
External column	203x203x46	4	842.26	0	842.26	877	738.66
Internal column	203x203x46	2	421.13	0	421.13	877	369.33
Column base plate	400x400x20	3	0	75.36	75.36	901	67.90
Total					2758.16		2339.71

Table 2.21(b) Calculation of percentage in weight and cost savings for flush end plate partial strength joint for 2 bay, 2 storey, and 6m span frame.

			Weight in (kg)			Costing in pounds	
Component	Section	Number	Shaft	Fittings	Total	Rate per 1000 Kg	Total
Roof beam	305x127x42	2	486.94	26.75	513.69	880	452.05
Floor beam	356x171x57	2	660.84	30.60	691.44	849	587.03
External column	203x203x46	4	844.56	0	844.56	877	740.68
Internal column	203x203x46	2	422.28	0	422.28	877	370.34
Column base plate	400x400x20	3	0	75.36	75.36	901	67.90
Total					2547.33		2218.00
Total mass different					210.83	Total cost different	121.71
Percentage weight saving					7.64%	Percentage cost savings	5.20%

Table 2.22 Additional increase in percentage savings due to effect of increasing the number of bays from two to four.

	S275 steel	S355 steel
Flush end plate connection	-1.30% to 0.88% (Average =0.31%)	-1.80% to 0.71% (Average = 0.009%
Extended end plate connection	0.23% to 5.24% (Average = 1.54%	0.21% to 1.65% (Average = 0.63%)

Table 2.23 Range of percentage savings due to effect of using S355 steel.

	S275 steel	S355 steel
Flush end plate connection	4.85% to 10.33% (Average = 7.81%)	1.14% to 8.95% (Average =4.90%)
Extended end plate connection	2.38% to 11.95% (Average = 8.54%)	-0.56% to 8.20% (Average = 2.12%)

Table 2.24(a) Range of percentage of savings and the average values due to effect of increasing the beam span from 6m to 9m.

	S275 steel		S355 steel	
	6m	9m	6m	9m
Flush end plate connection	5.63% to 10.33% (Average = 8.12%)	4.85% to 9.13% (Average = 7.24%)	3.22% to 8.95% (Average = 5.79%)	1.14% to 7.26% (Average = 4.01%)
Extended end plate connection	6.76% to 11.95% (Average = 10.39%)	2.38% to 10.73% (Average = 6.68%)	2.44% to 8.20% (Average = 4.97%)	-0.56% to 4.98% (Average = 2.88%)

Table 2.24(b) Ratio of moment resistance of connection to maximum design moment for beam in simple constrution.

	S275 steel			
	6m		9m	
	Min.Weight	Min. Depth	Min. Weight	Min. Depth
Flush end plate connection	0.22 to 0.27	0.19 to 0.23	0.13 to 0.16	0.13 to 0.15
Extended end plate connection	0.38 to 0.50	0.33 to 0.35	0.23 to 0.29	0.20 to 0.26

Table 2.25 Range of percentage of savings and the average values due to effect on the use of extended end plate connections.

	S275 steel	S355 steel
Flush end plate connection	5.63% to 10.33% (Average = 7.68%)	1.14% to 8.95% (Average =4.90%)
Extended end plate connection	2.38% to 11.95% (Average = 8.54%)	-0.56% to 8.20% (Average = 3.92%)

Table 2.26 Range of percentage of savings and the average values due to effect of selecting beam sections according to minimum weight instead of minimum depth.

	S275 steel		S355 steel	
	Min. weight	Min. depth	Min. weight	Min. depth
Flush end plate connection	4.85% to 10.33% (Average = 8.13%)	5.63% to 9.13% (Average = 7.49%)	1.14% to 8.95% (Average = 4.98%)	3.22% to 7.26% (Average = 7.49%)
Extended end plate connection	4.01% to 11.95% (Average = 9.29%)	2.38% to 11.86% (Average = 4.58%)	-0.56% to 8.20% (Average = 4.18%)	2.48% to 4.98% (Average = 3.67%)

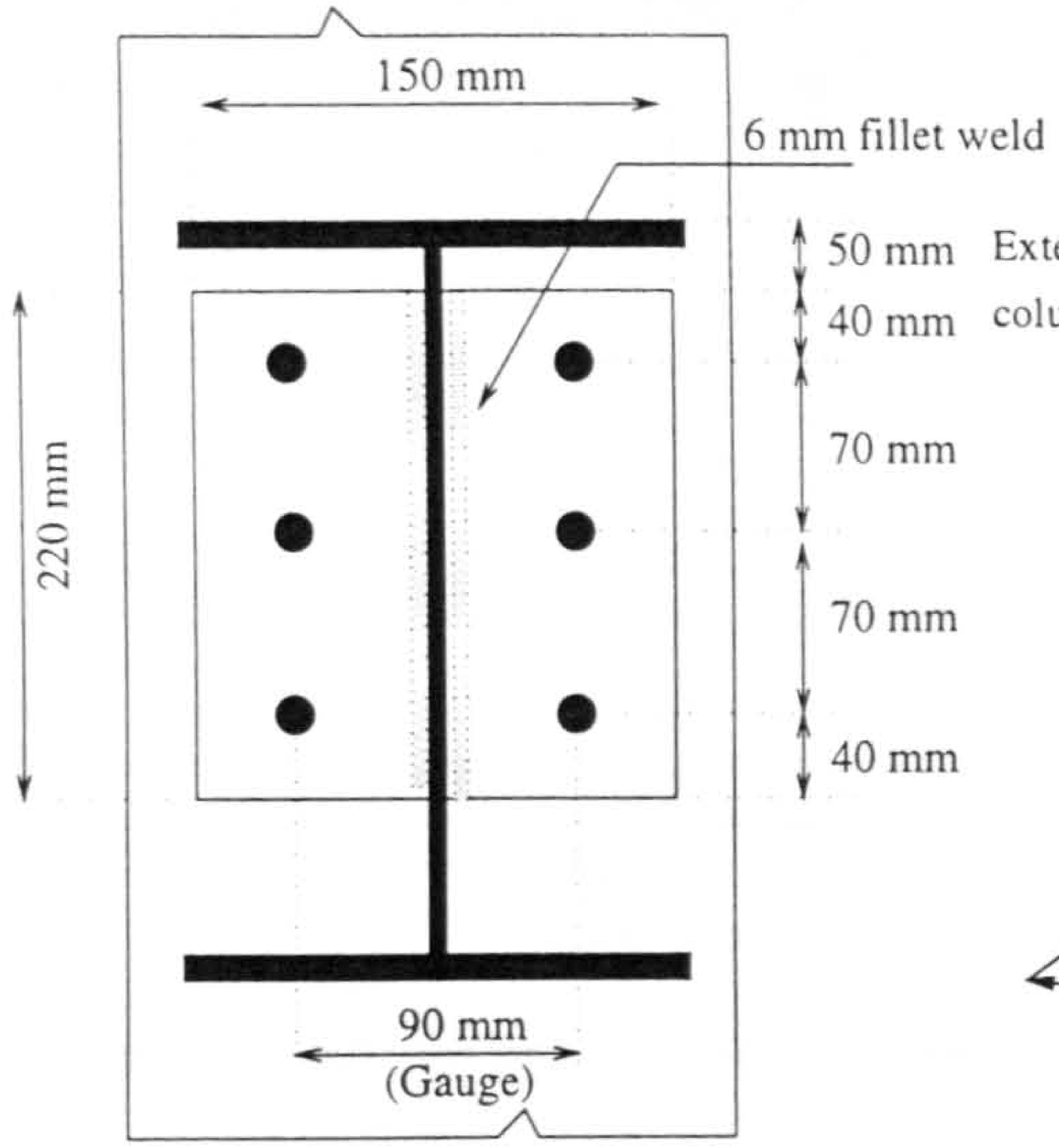


Figure 2.1 Typical flexible end plate connection

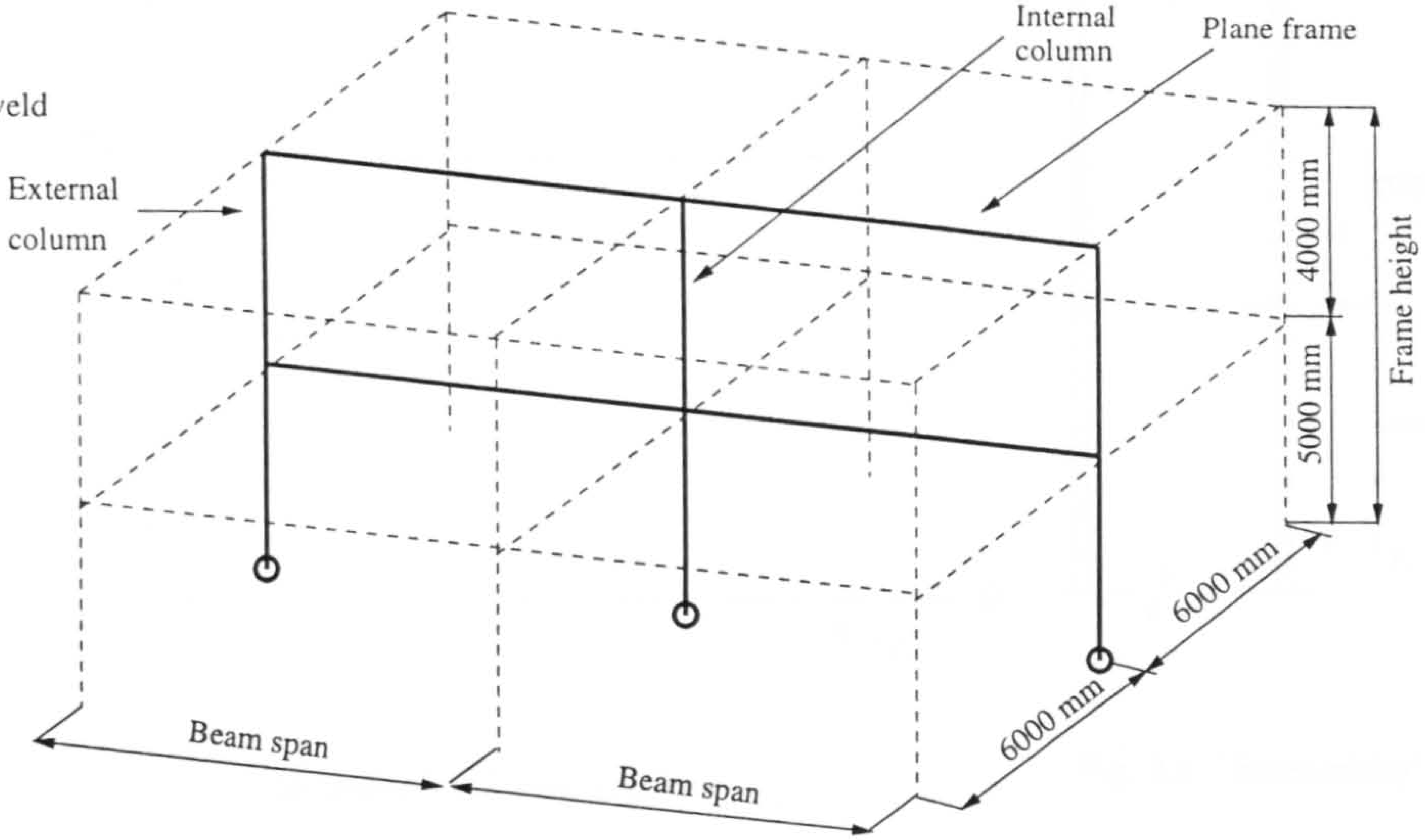


Figure 2.2 Typical frame layout

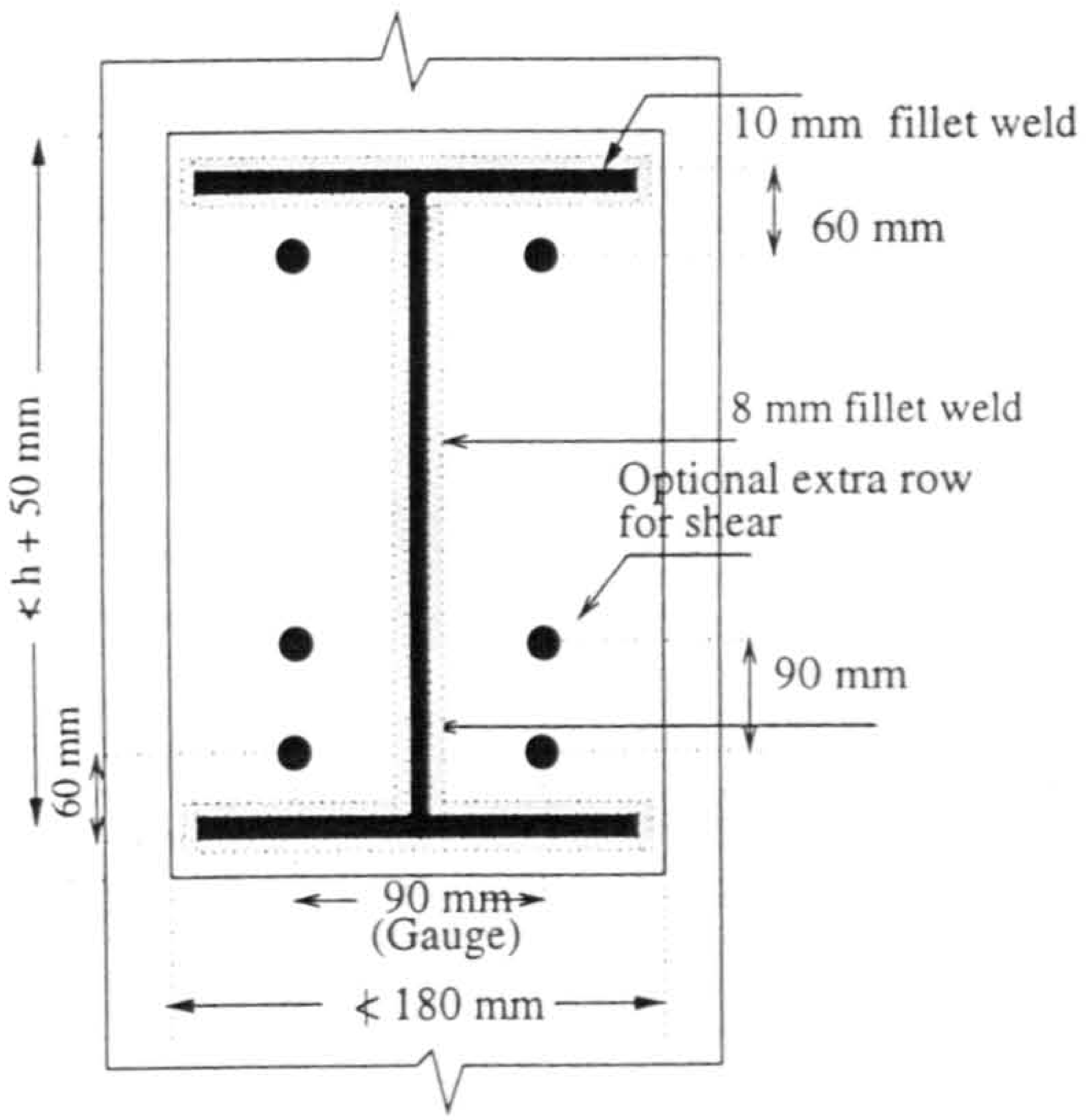


Figure 2.3(a) Typical flush end plate connection

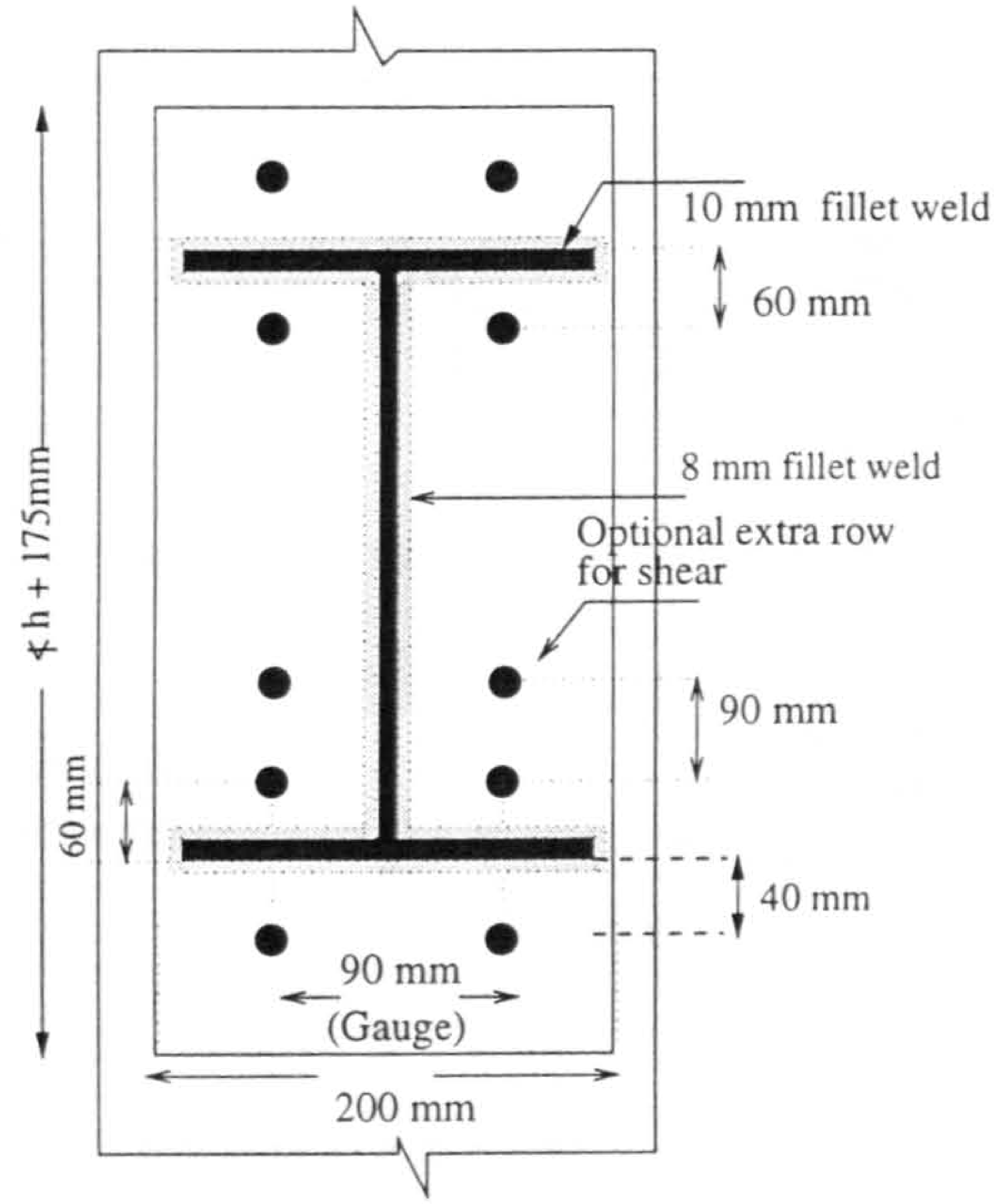


Figure 2.3(b)Typical extended end plate connection

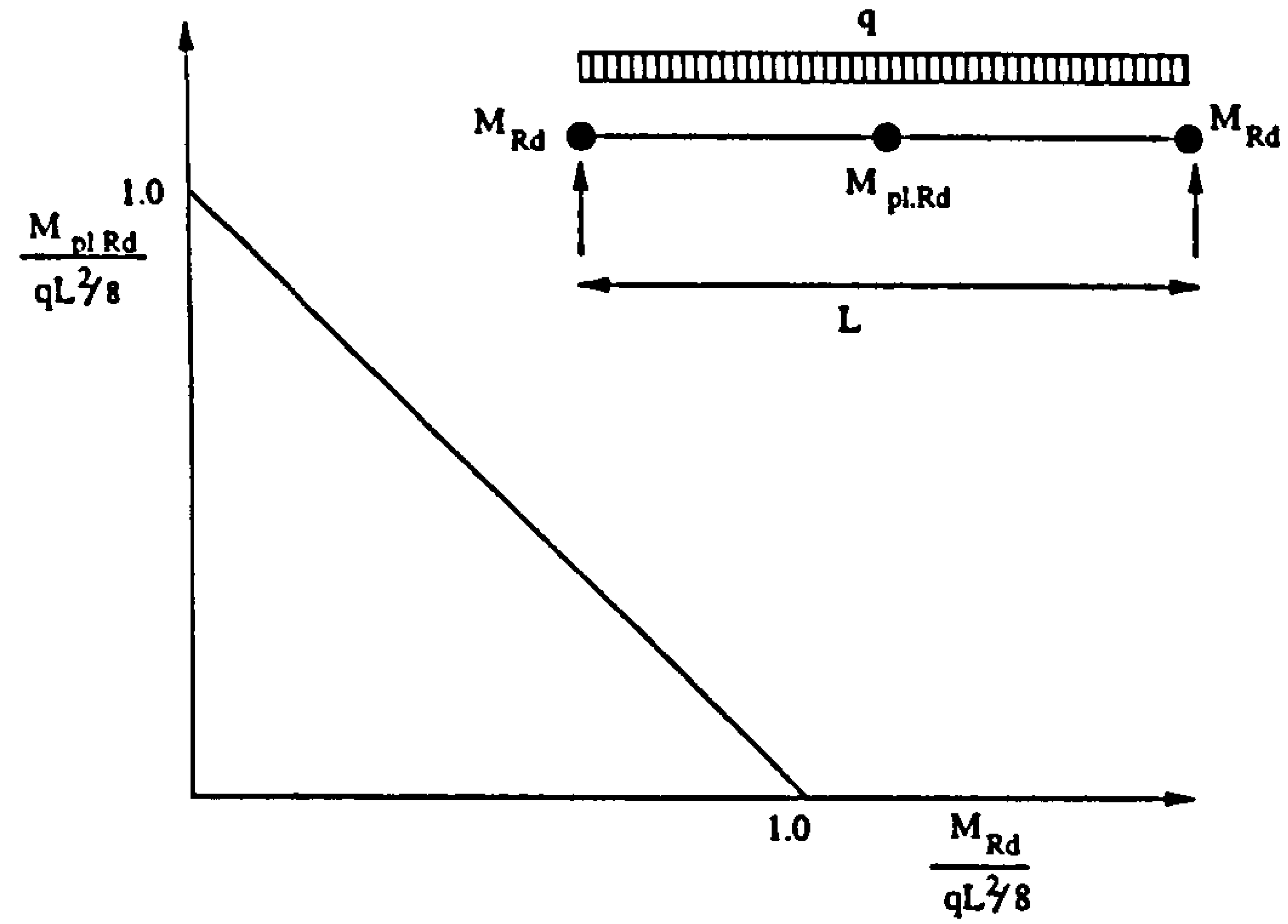


Fig. 2.4 Plastic analysis of beam with partial-strength connections.

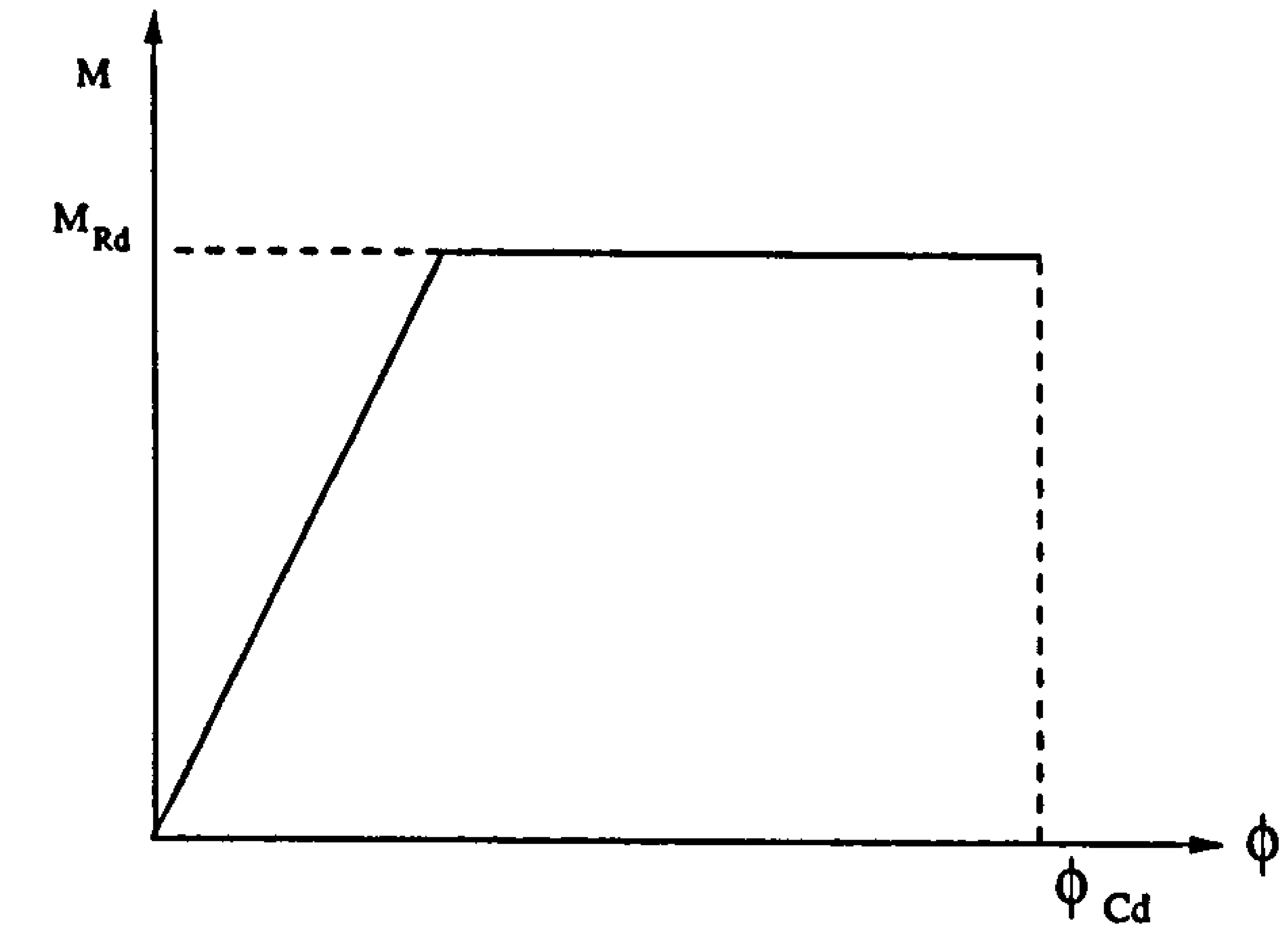


Fig. 2.5 Bi-linear M- ϕ characteristic.

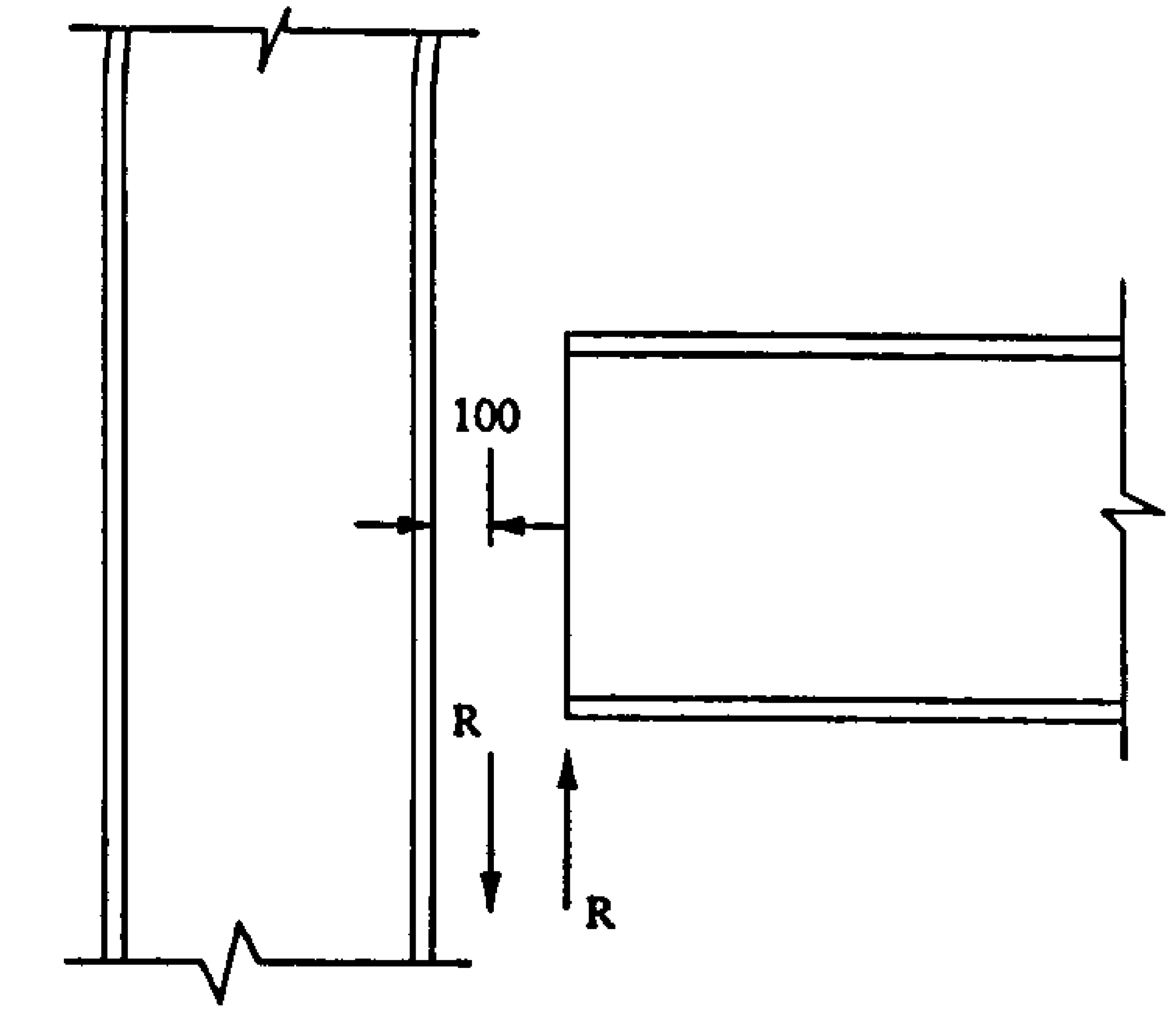


Fig. 2.6 "Eccentricity" moment

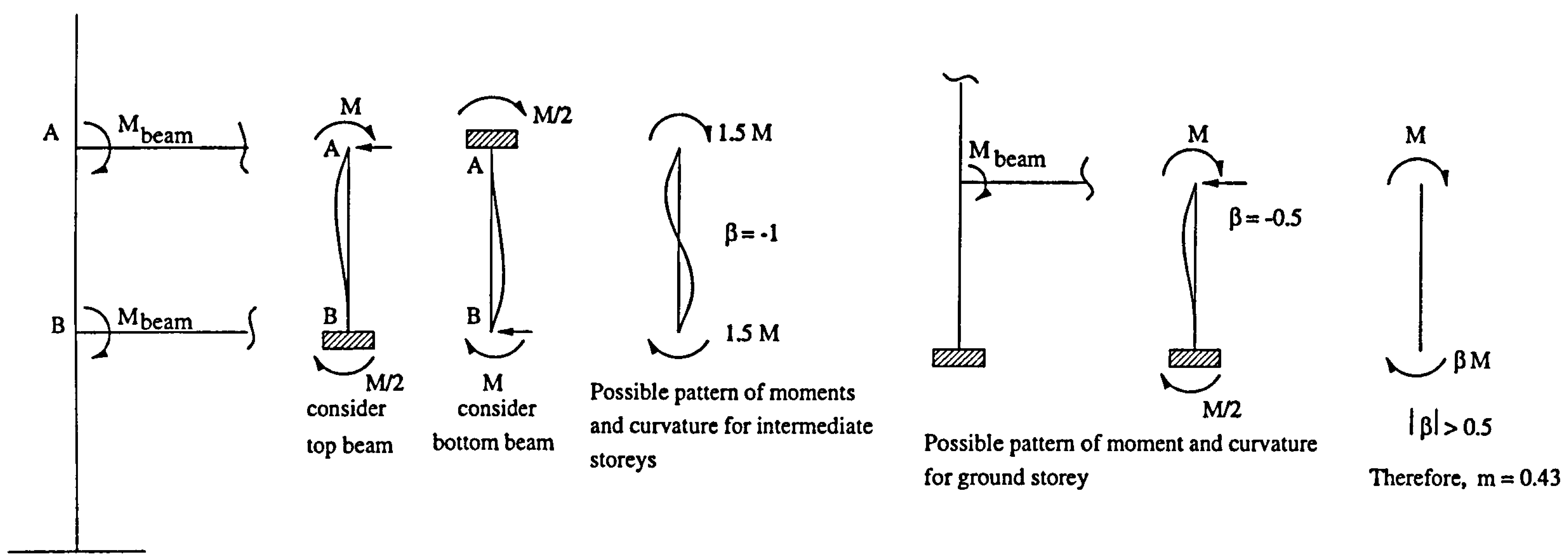


Fig. 2.7 Justification of moment factor m = 0.43

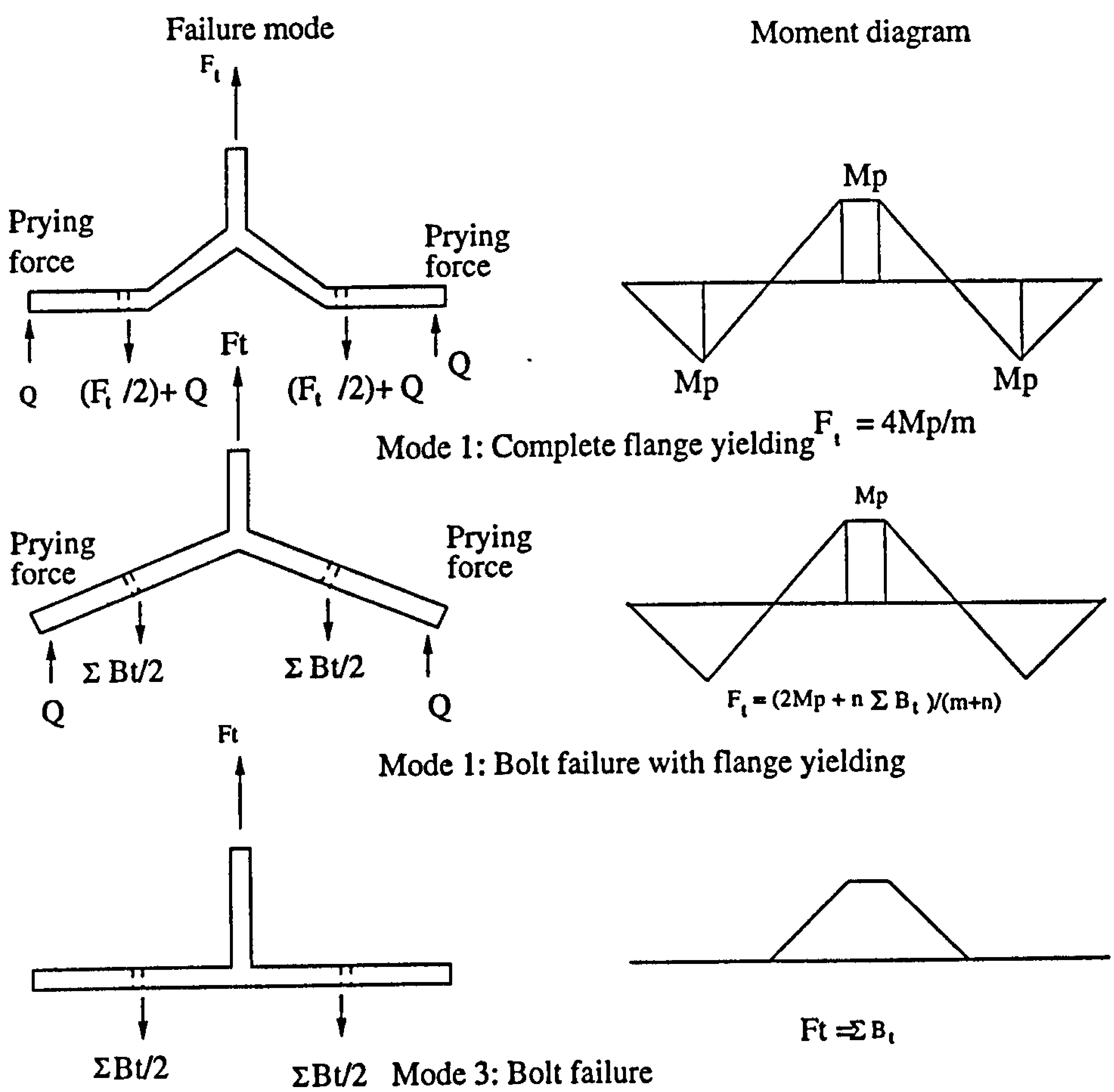


Figure 2.8: Failure modes of end plates

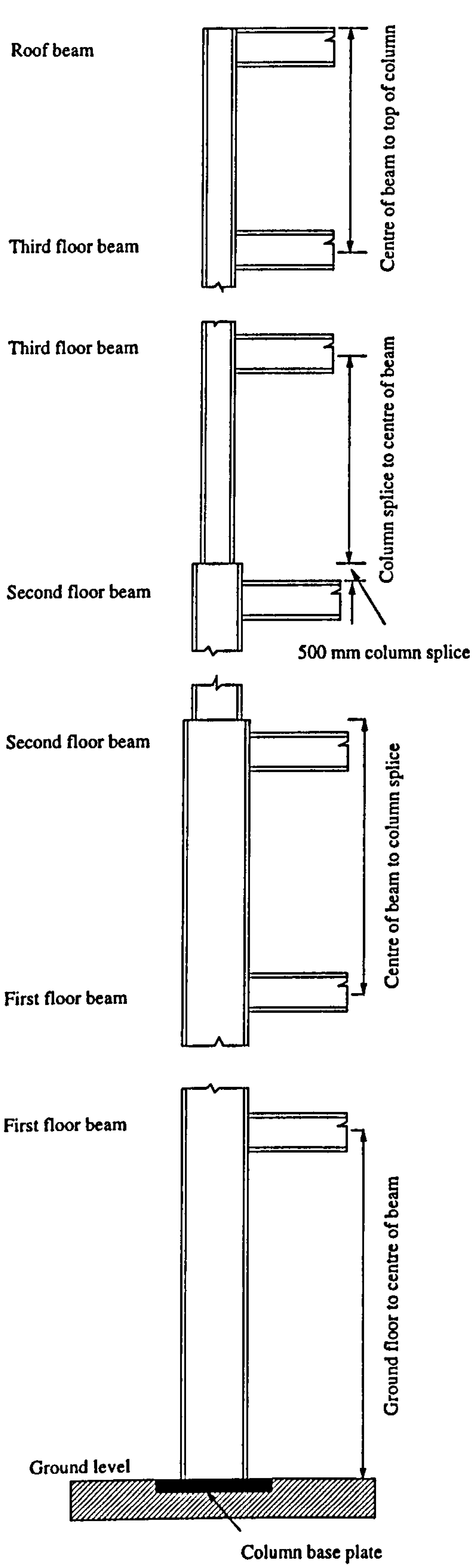


Figure 2.9 Dimension to calculate total mass of external column for flexible end plate connection

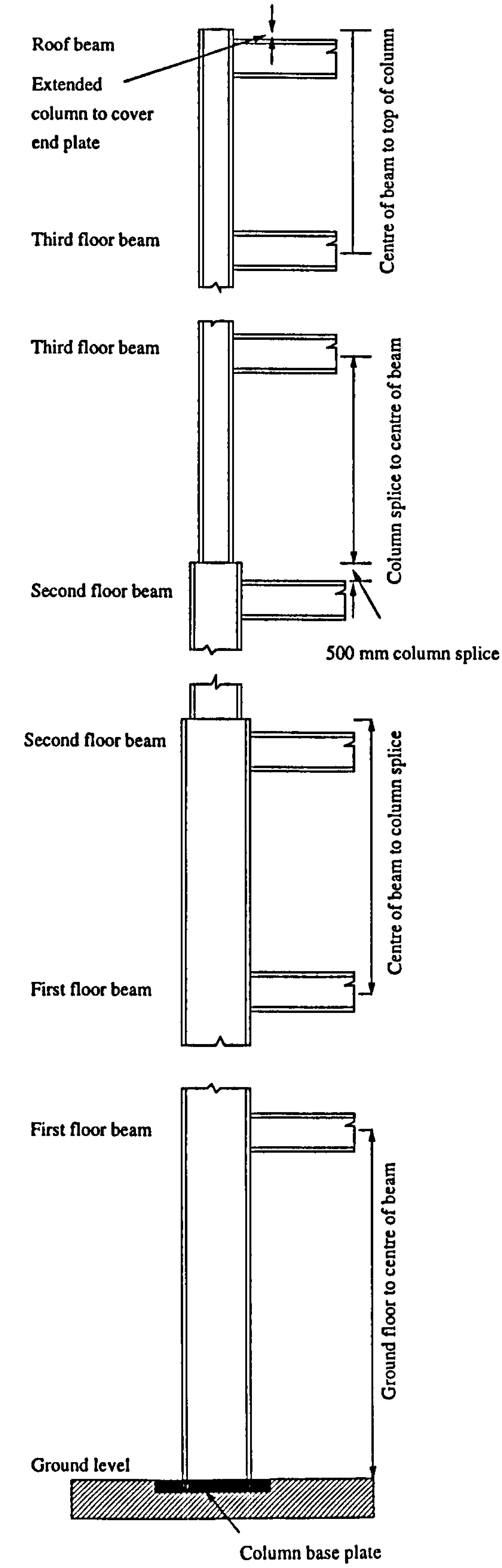


Figure 2.10 Dimension to calculate total mass of external column for flush end plate connection

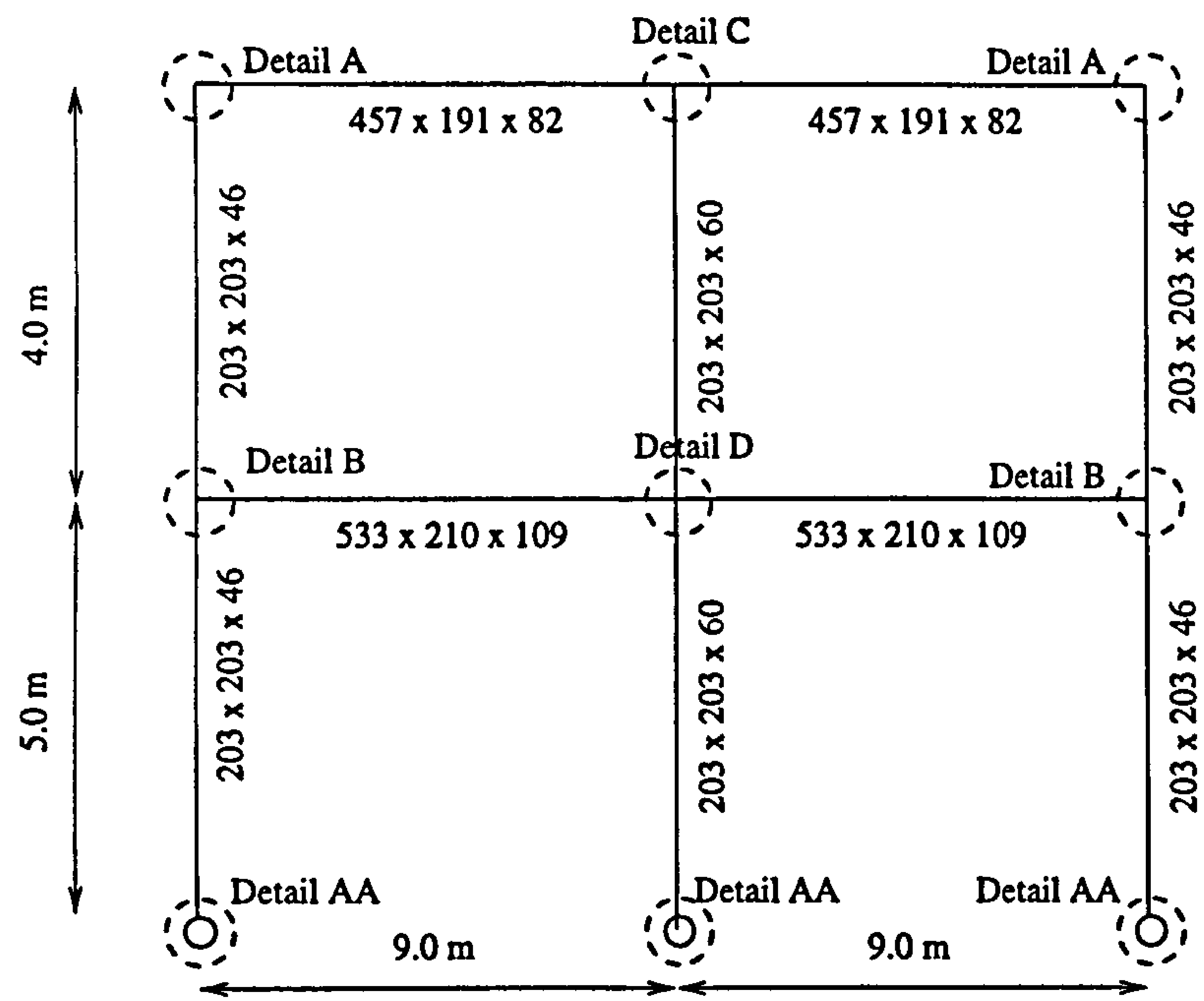


Fig. 2.11(a) 2 bay 2 storey braced frame designed for simple construction

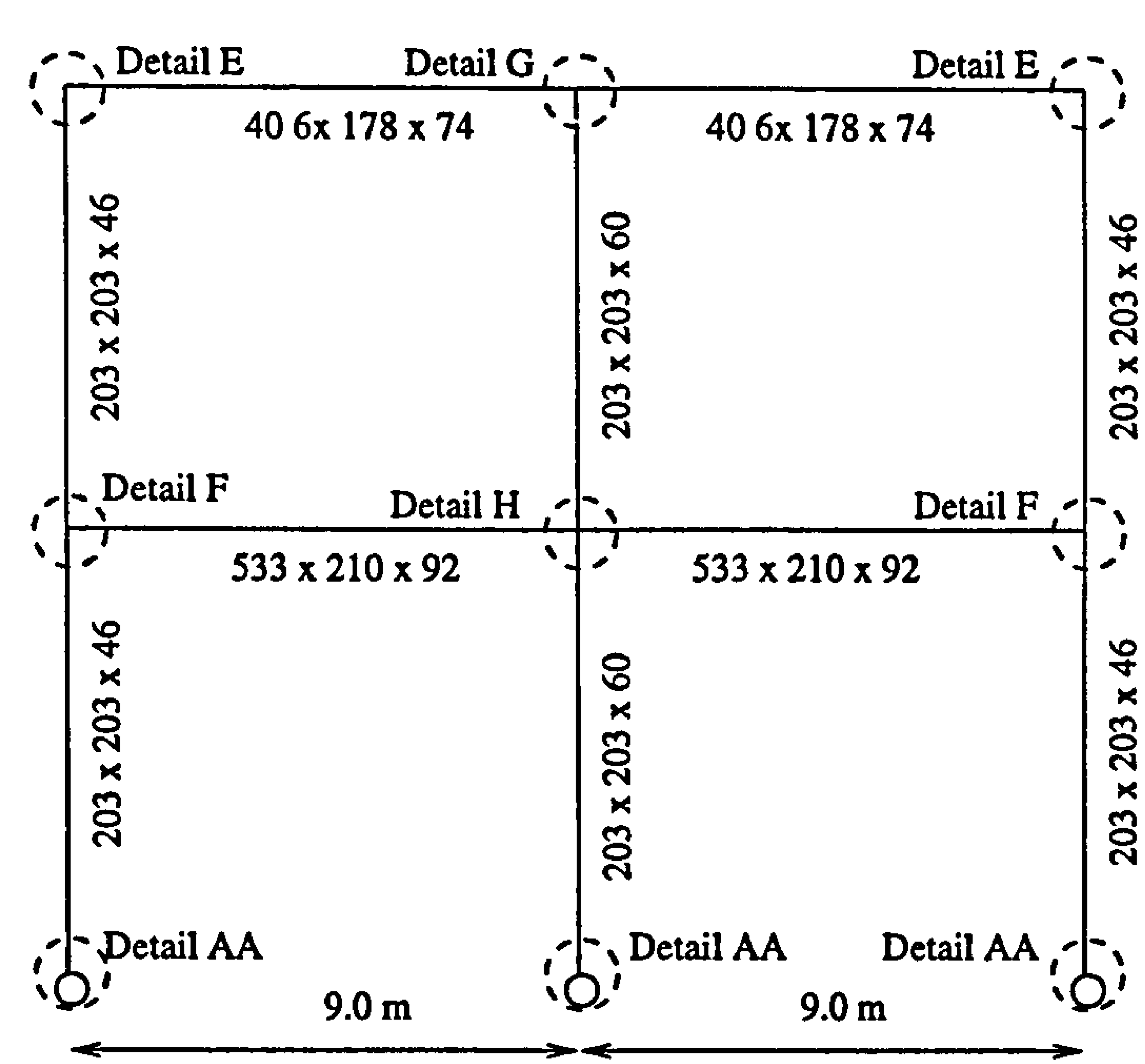
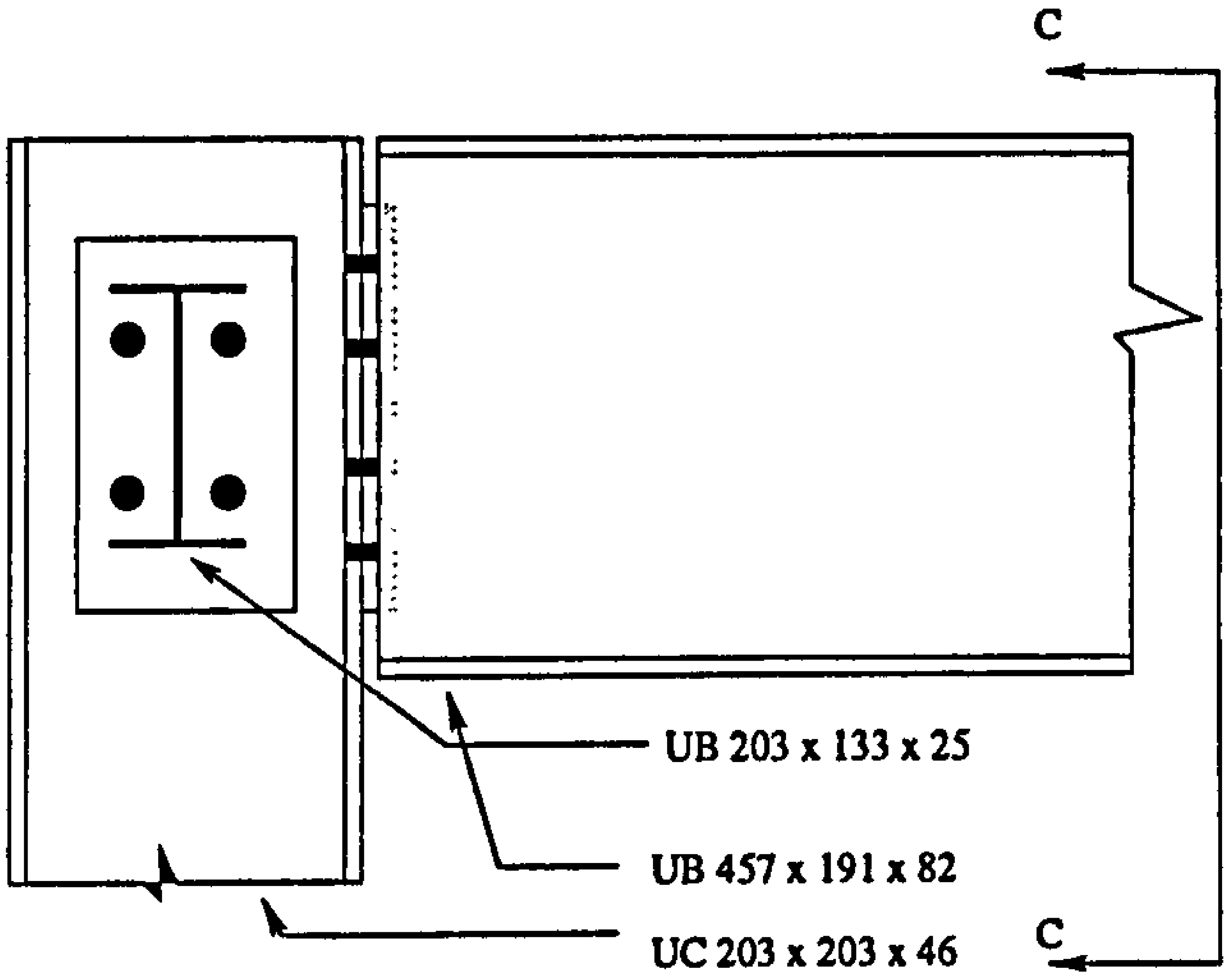


Fig 2.11 (b) 2 bay 2 storey braced frame designed for semi-continuous constrcution.

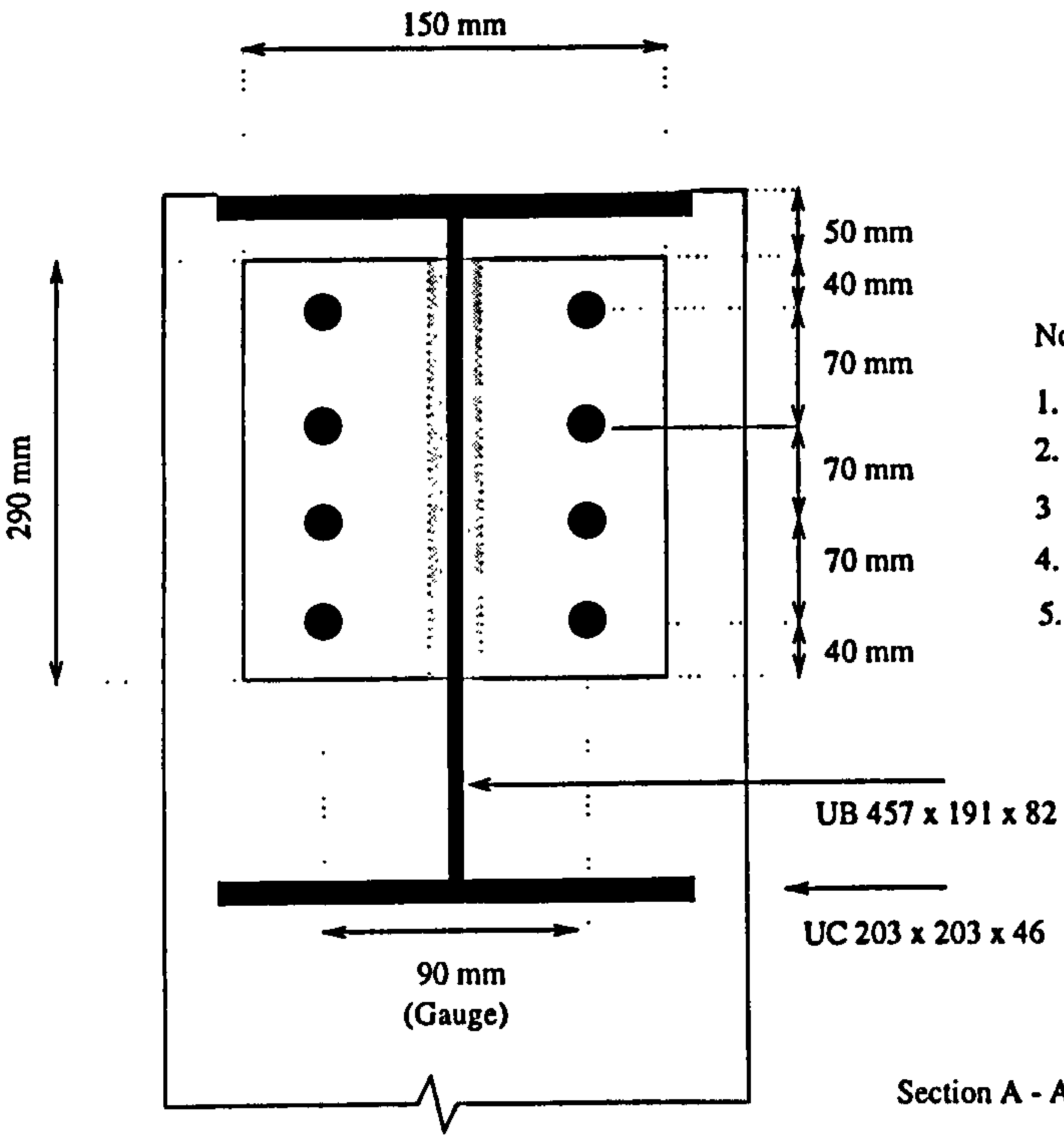
	Dead load	Live Load
Roof Beam	4.0 kN/m2	1.5 kN.m2
Floor Beam	4.0 kN/m2	4.0 kN/m2

Note:- All Steel Member: Grade 50

Beams designed for minimum depth



Detail A

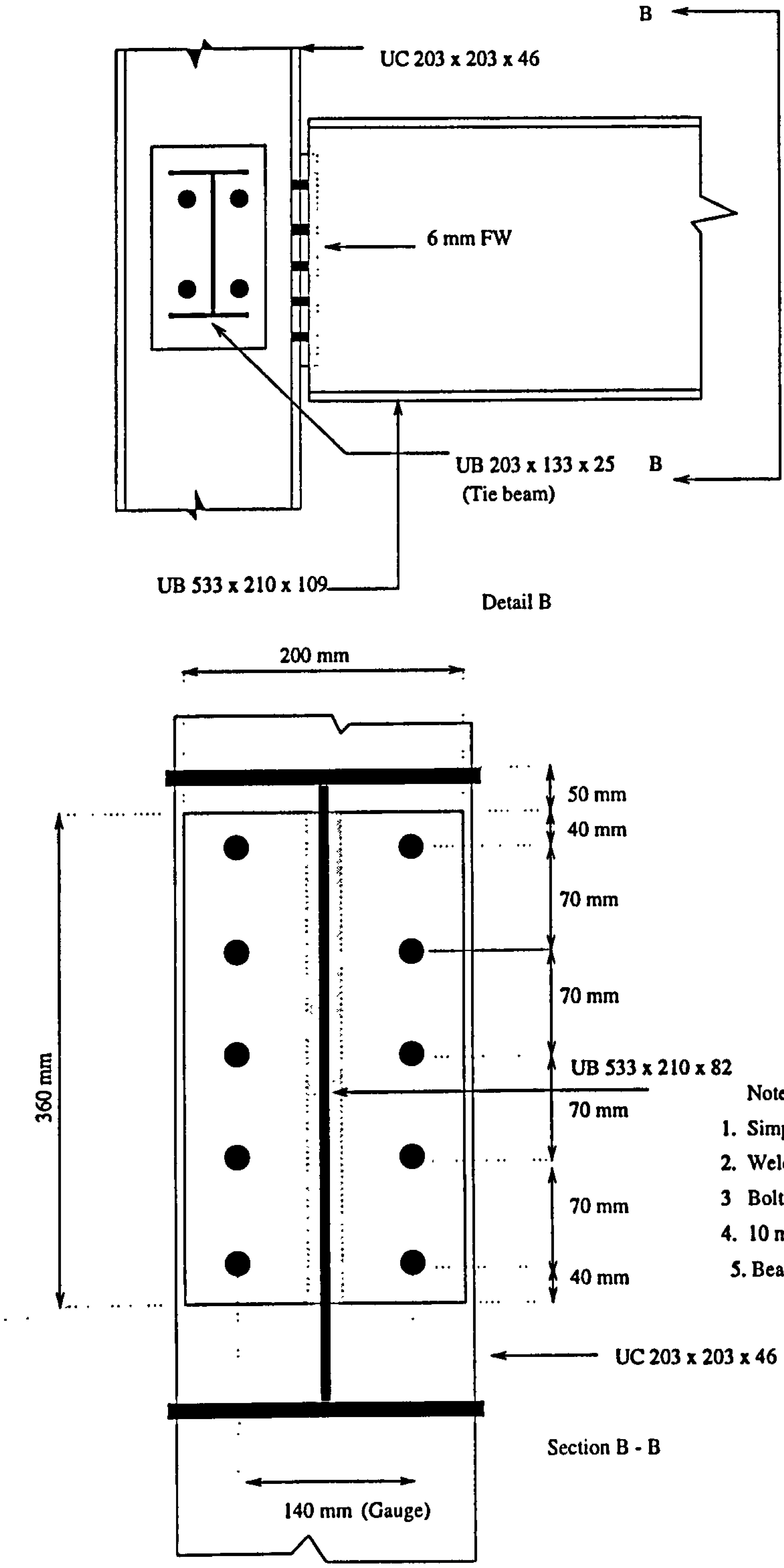


Note :-

1. Simple construction (Flexible end plates connection)
2. Web to end plate welds 6 mm FW
3 Bolt size 20 mm diameter bolt grade 8.8
4. 8 mm thick end plate (S275)
5. Beams designed for minimum depth

Section A - A

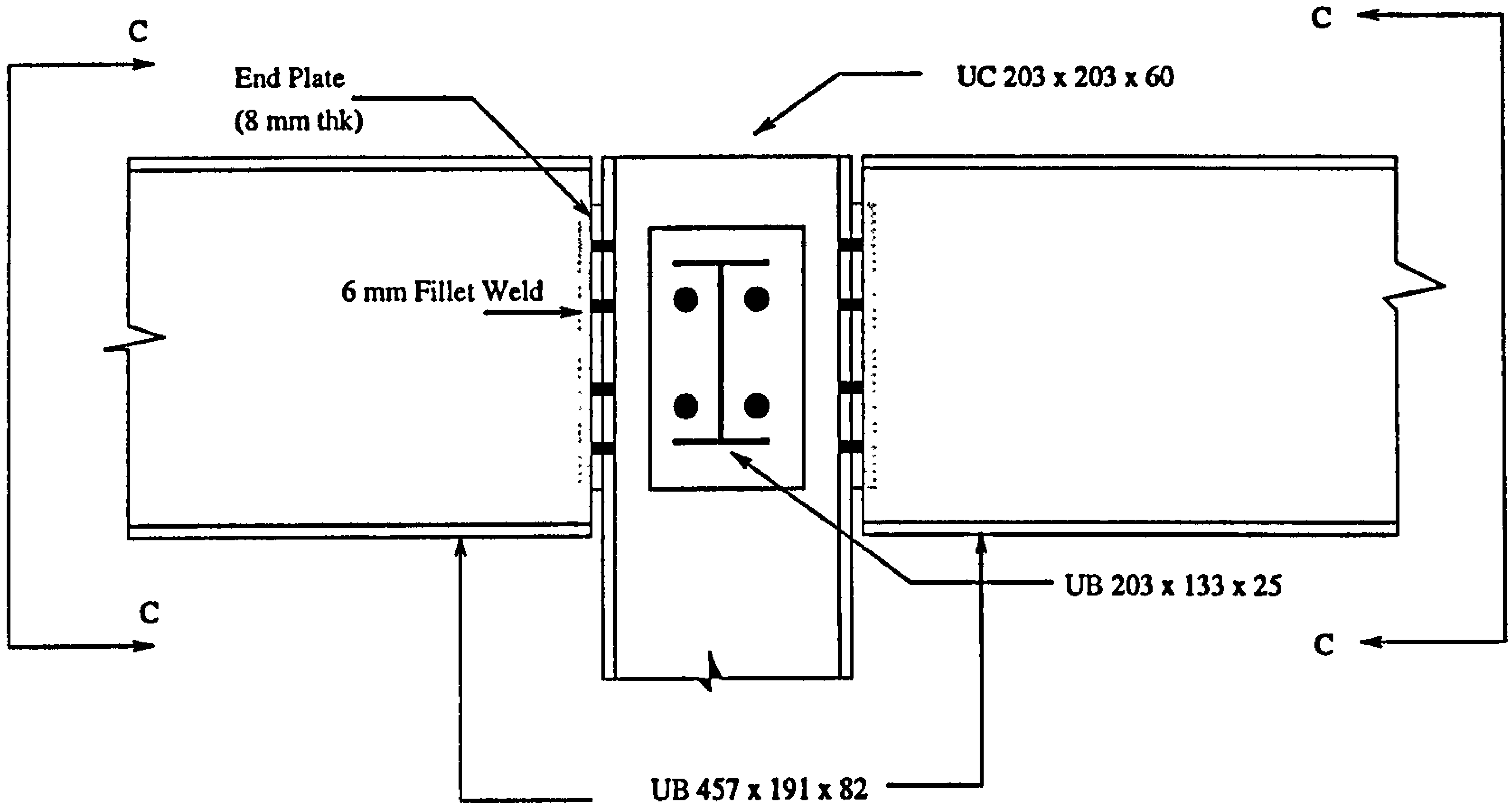
Fig 2.12 Detail of connection for roof beam to external column



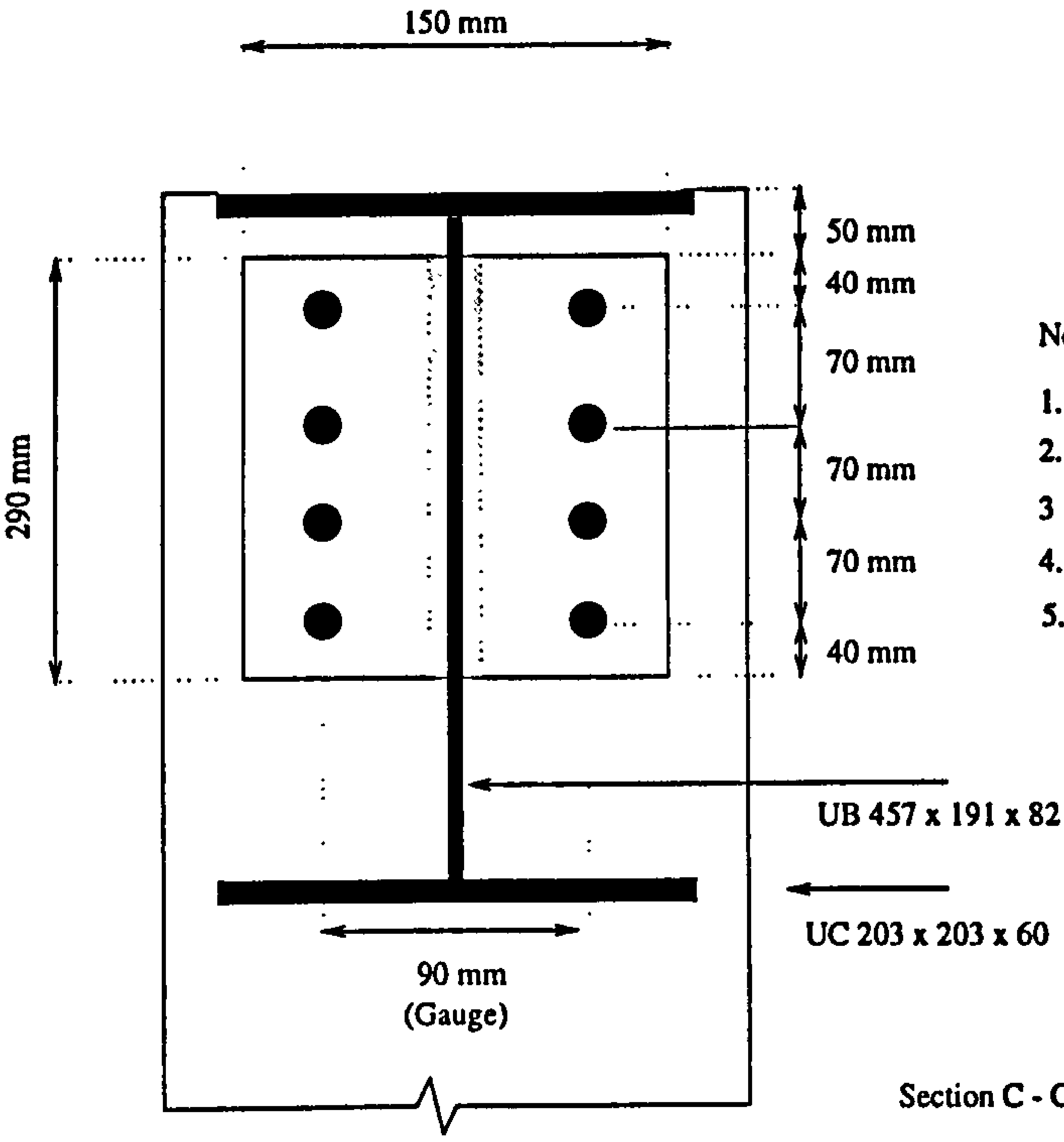
Note :-

1. Simple construction (Flexible end plate connection)
2. Weld Size 6 mm Fillet Weld.
3. Bolt size 20 mm diameter bolt grade 8.8
4. 10 mm thick end plate (S275)
5. Beams designed for minimum depth

Fig 2.13 Detail of connection for floor beam to external column



Detail C

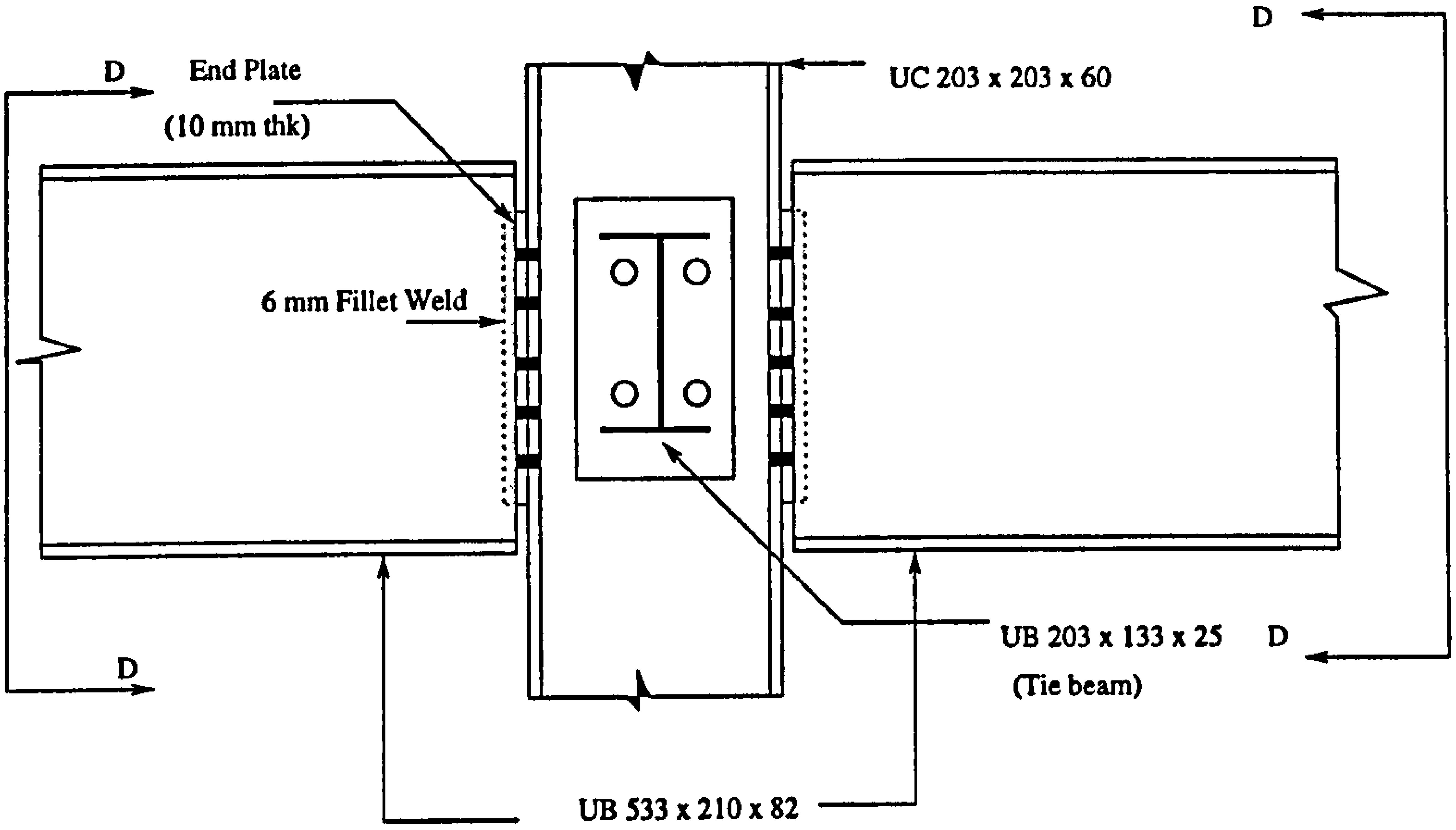


Note :-

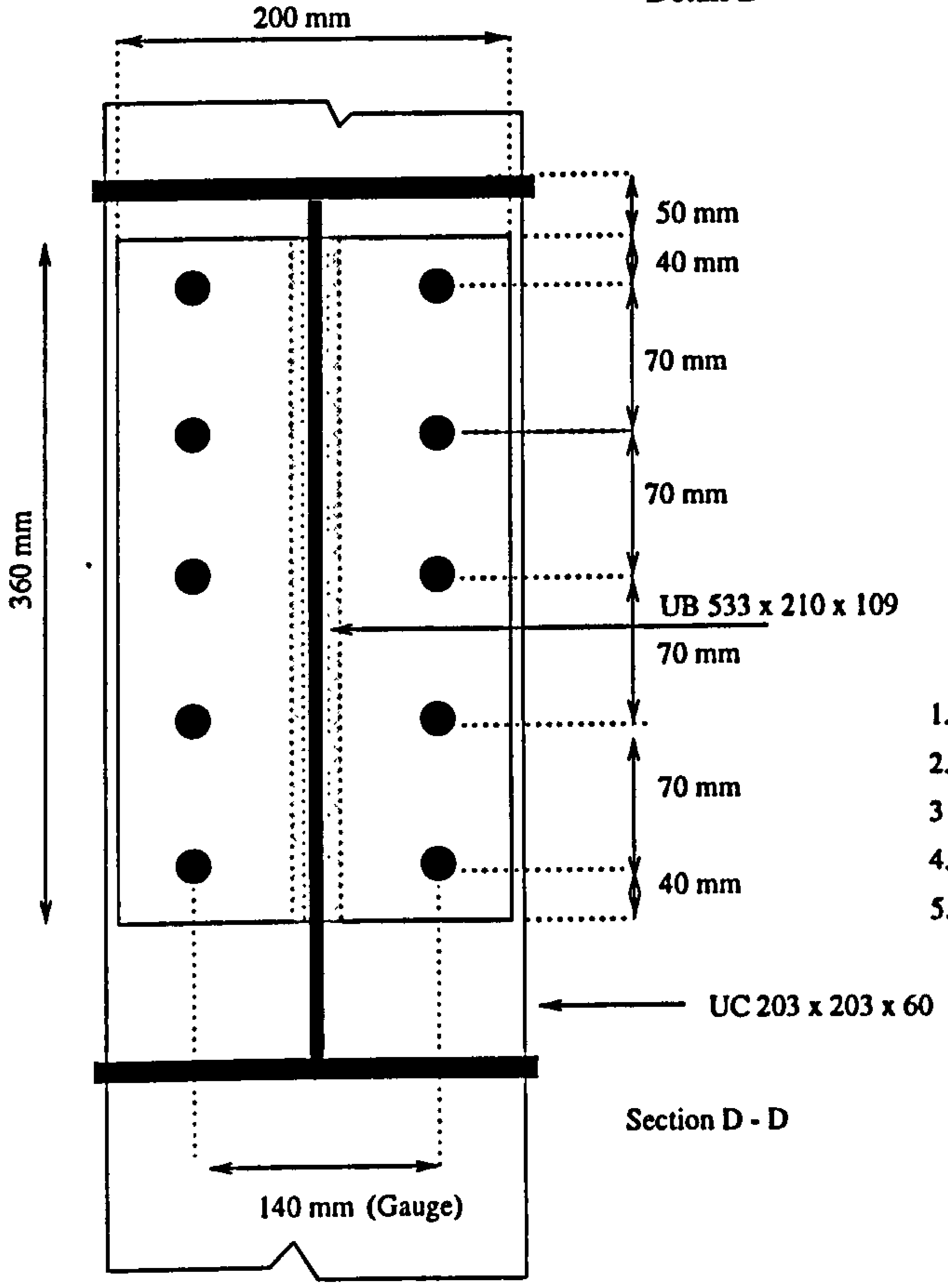
1. Simple construction (Flexible end plates connection)
2. Web to end plate welds 6 mm FW
3 Bolt size 20 mm diameter bolt grade 8.8
4. 8 mm thick end plate (S275)
5. Beams designed for minimum depth

Section C - C

Fig 2.14 Detail of connection for roof beam to internal column



Detail D

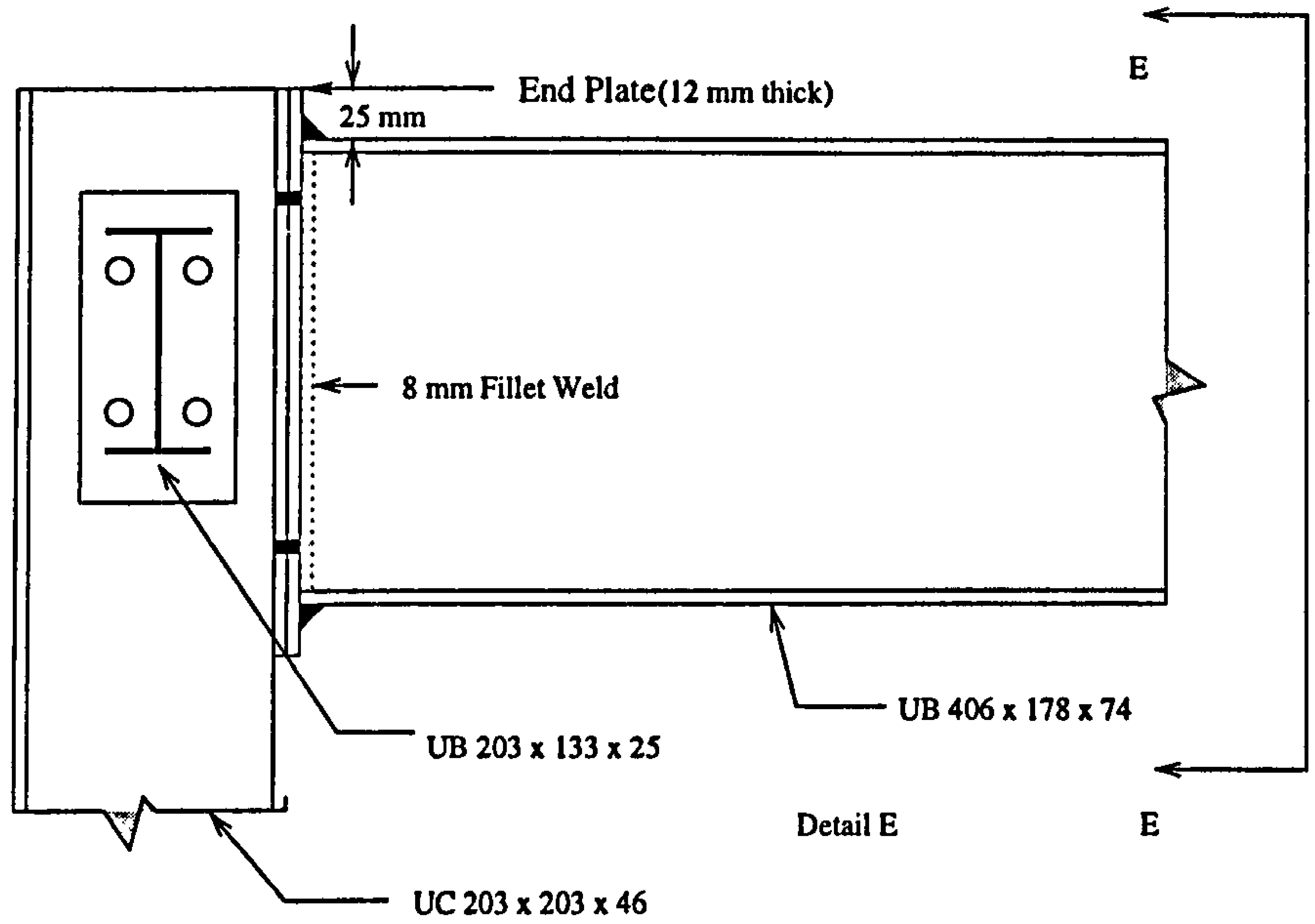


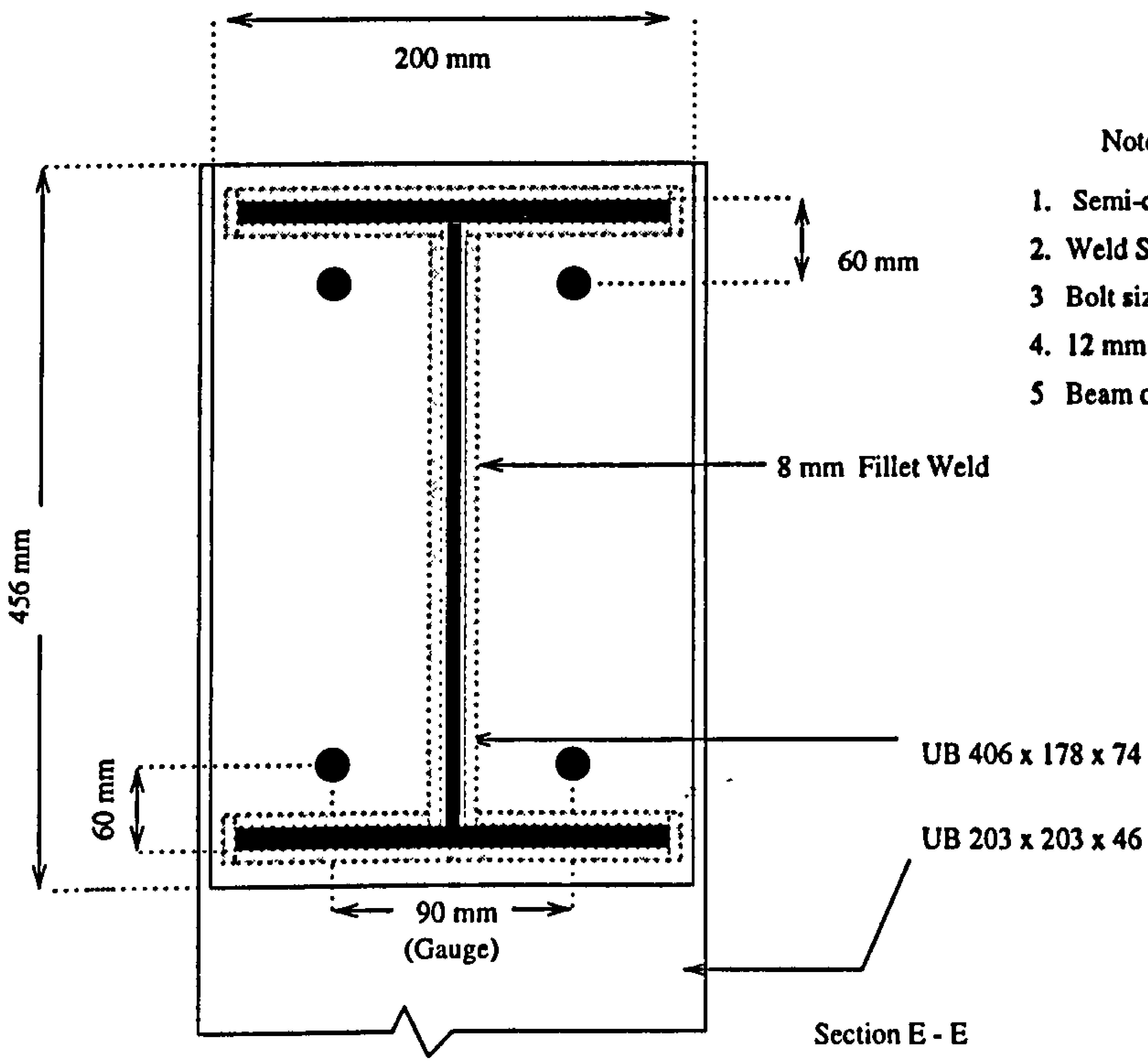
Section D - D

Note :-

1. Simple construction (Flexible end plate connection)
2. Weld Size 6 mm Fillet Weld.
3 Bolt size 20 mm diameter bolt grade 8.8
4. 10 mm thick end plate (S275)
5. Beams designed for minimum depth

Fig 2.15 Detail of connection for floor beam to internal column





Note :-

1. Semi-continuous construction (Flush end plate connection)
2. Weld Size 8 mm for web and 10 mm for flanges (FW).
3 Bolt size 20 mm diameter bolt grade 8.8
4. 12 mm thick end plate (S275)
5 Beam designed for minimum depth

Fig. 2.16 Detail of connection for roof beam to external column.

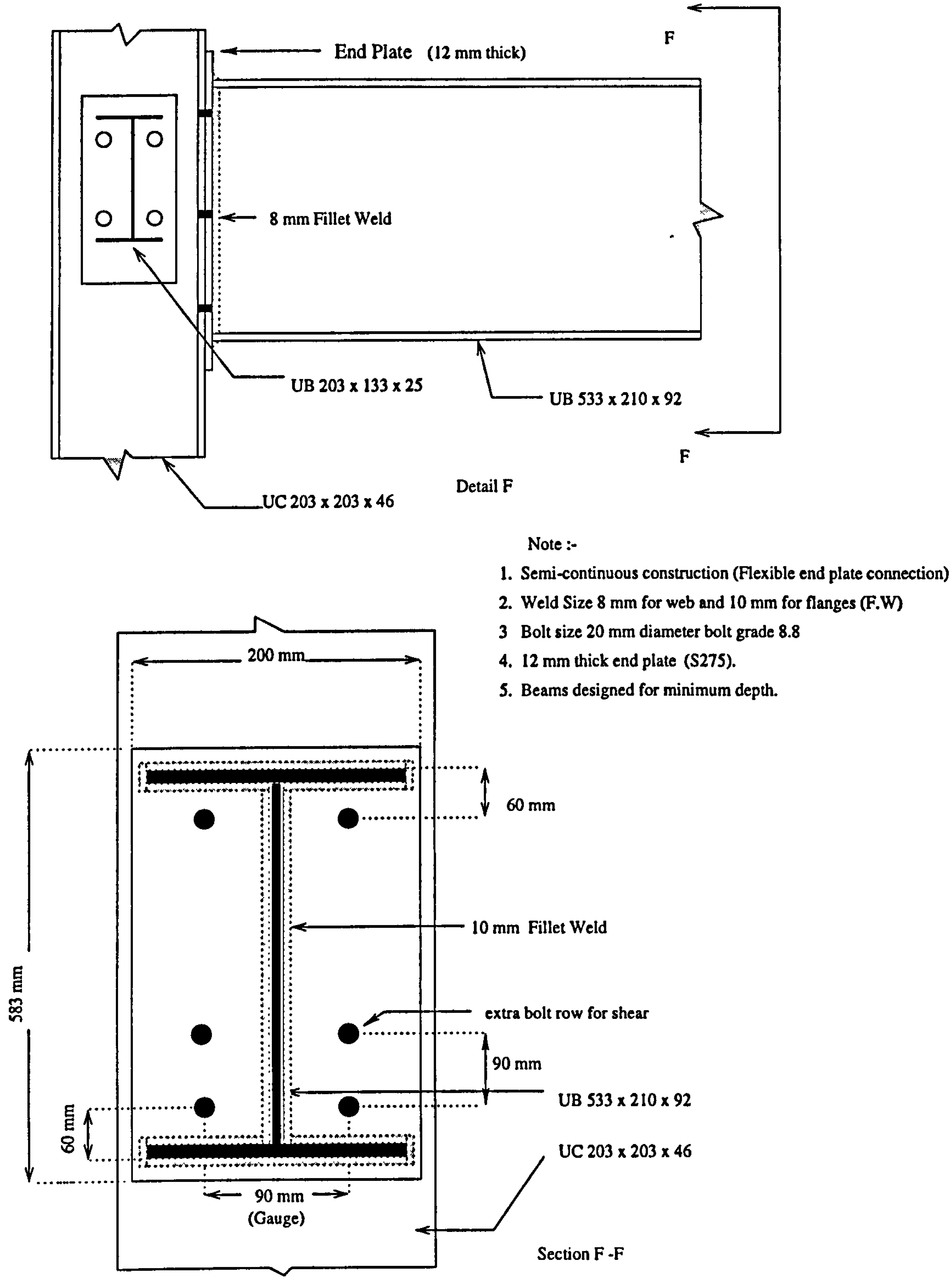
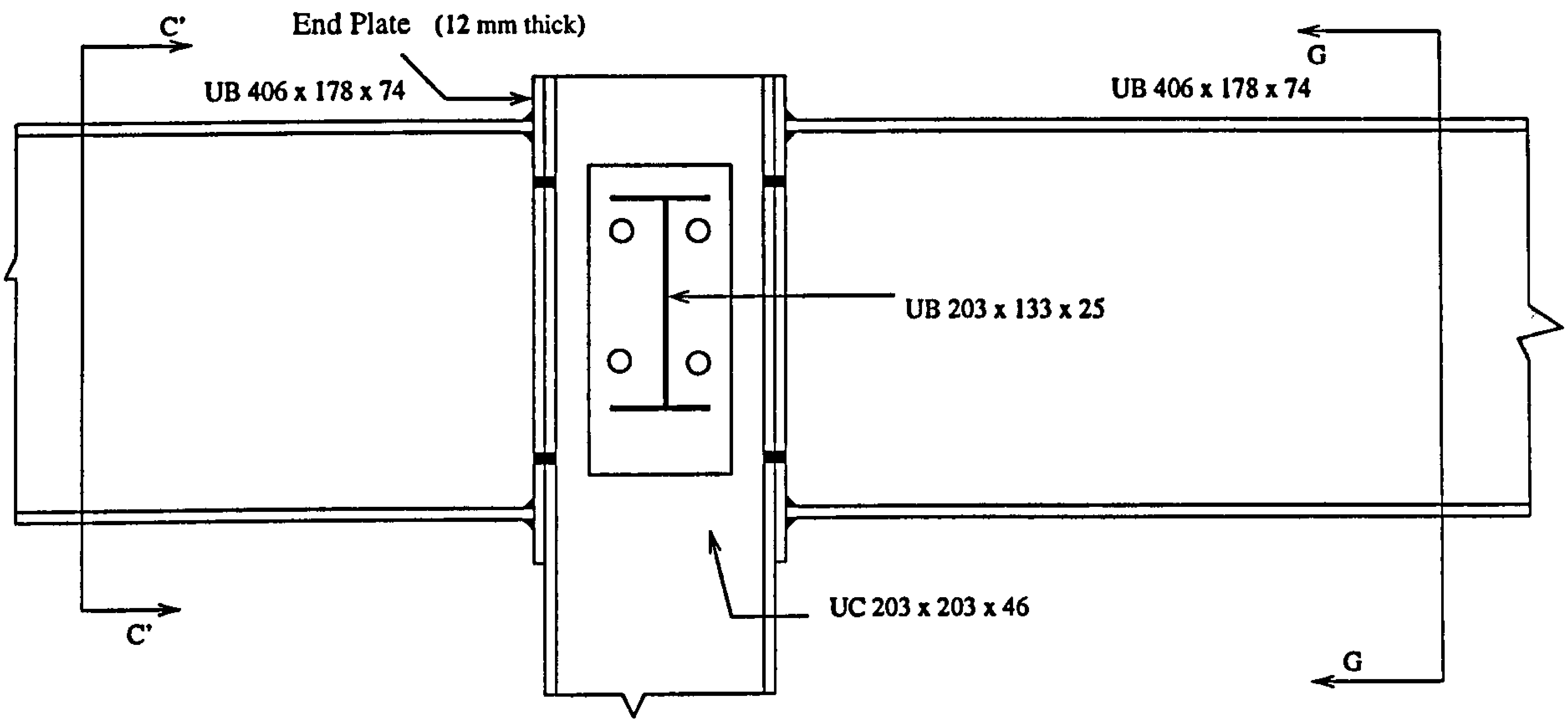
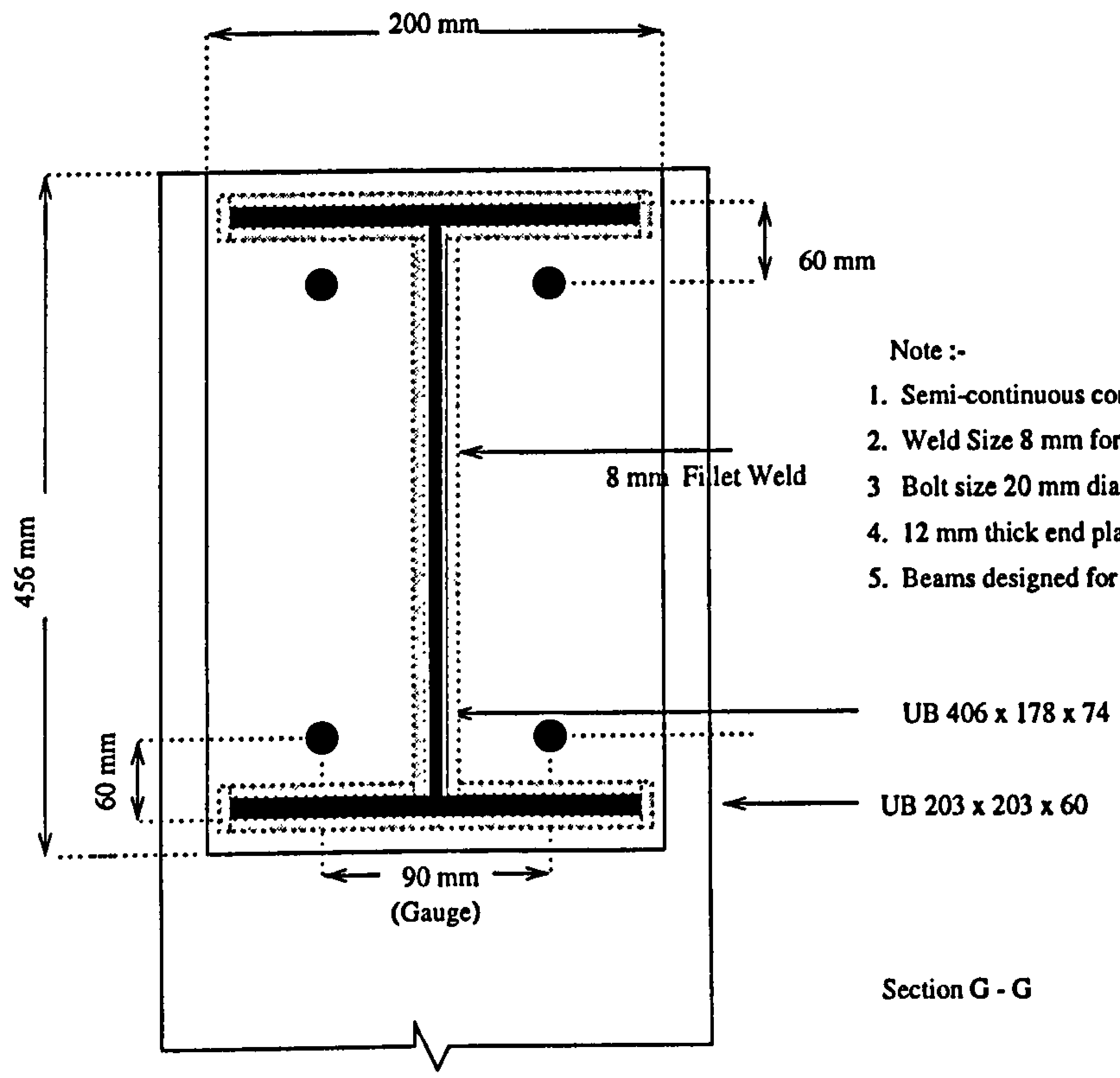


Fig. 2.17 Detail of connection for floor beam to external column.



Detail G



Note :-

1. Semi-continuous construction (Flush end plate connection)
2. Weld Size 8 mm for web and 10 mm for flanges (F.W).
3 Bolt size 20 mm diameter bolt grade 8.8
4. 12 mm thick end plate (S275)
5. Beams designed for minimum depth

Section G - G

Fig. 2.18 Detail of connection for roof beam to internal column .

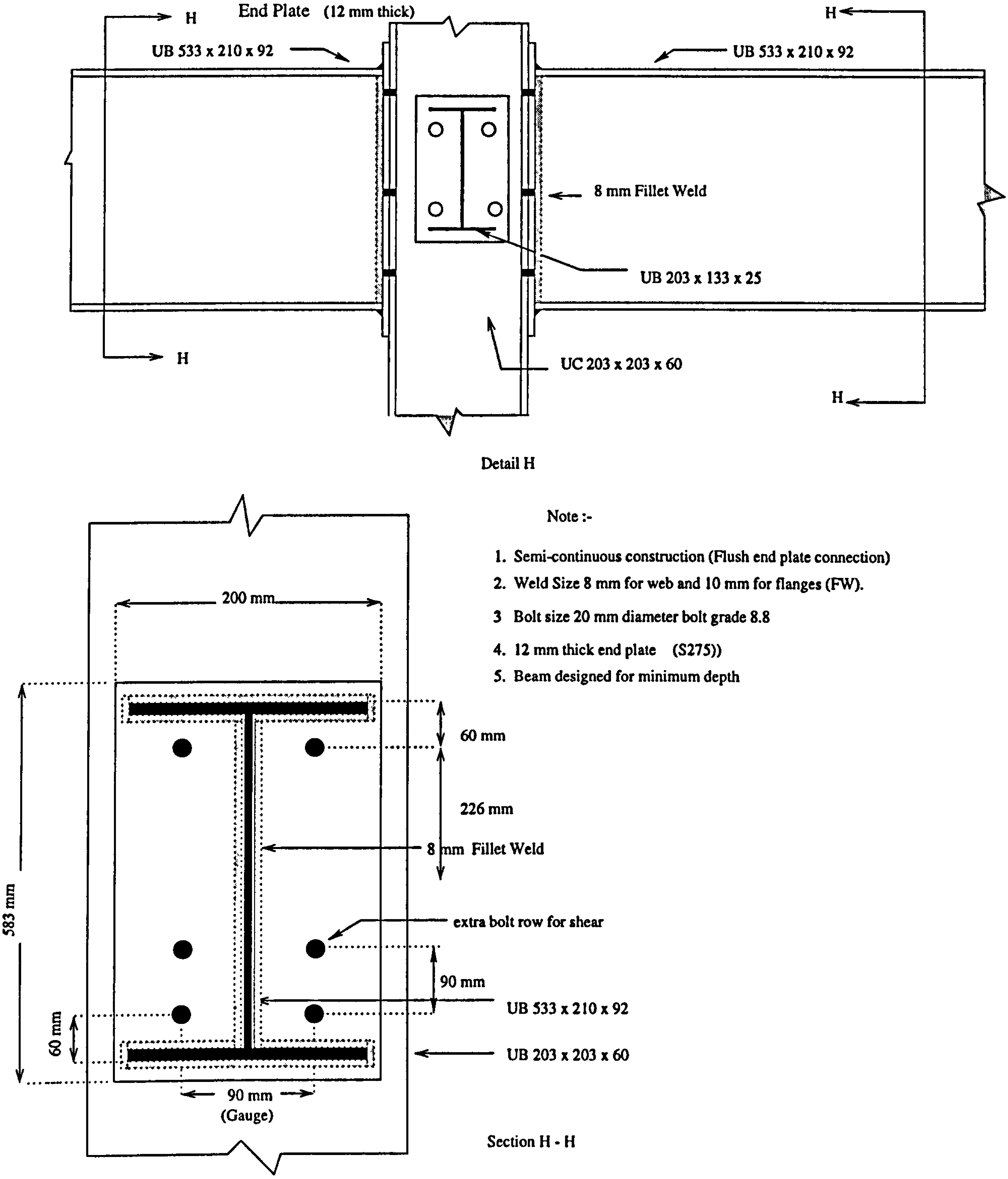


Fig. 2.19 Detail of connection for floor beam to internal column.

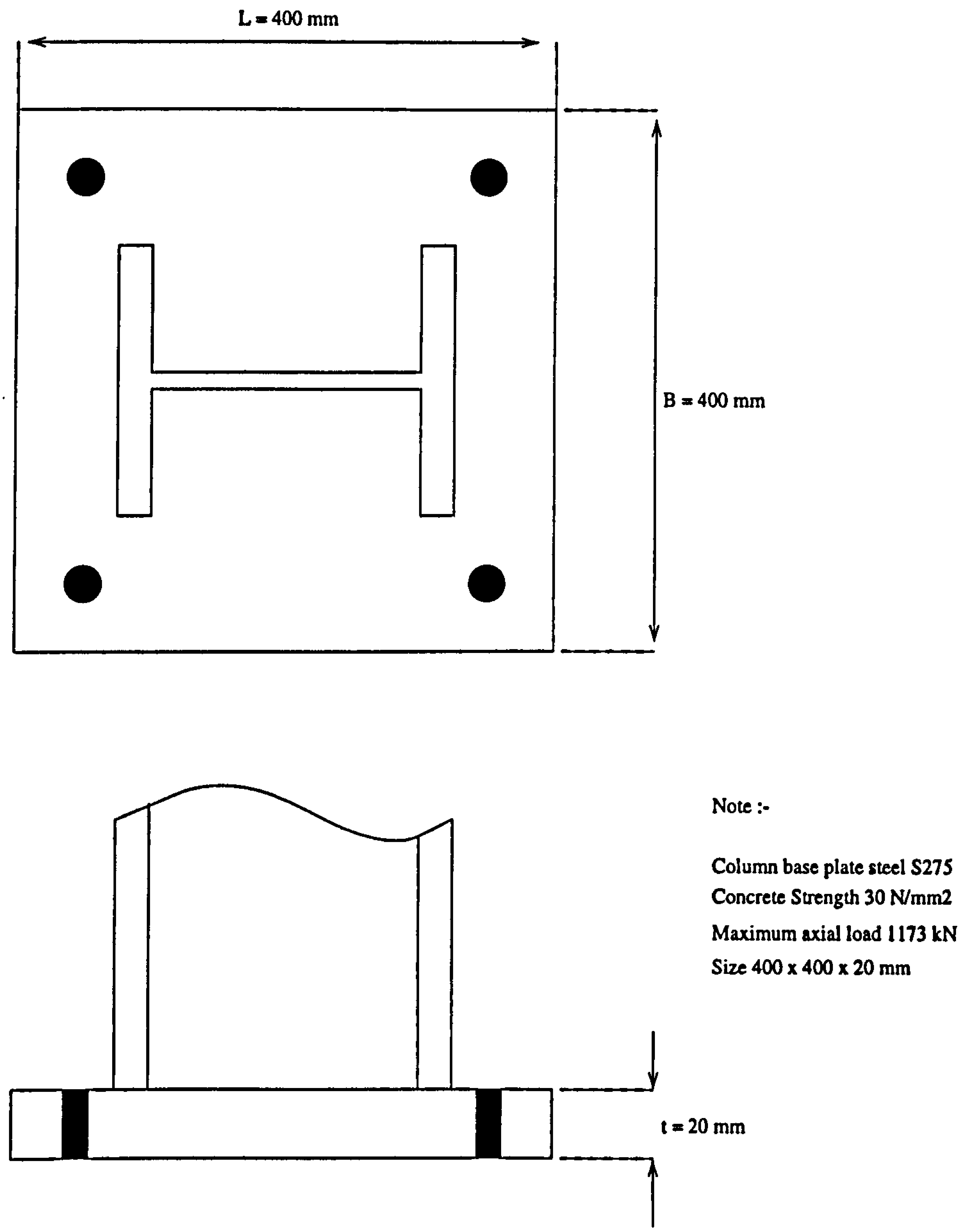


Fig. 2.20 Column base plate for detail AA.

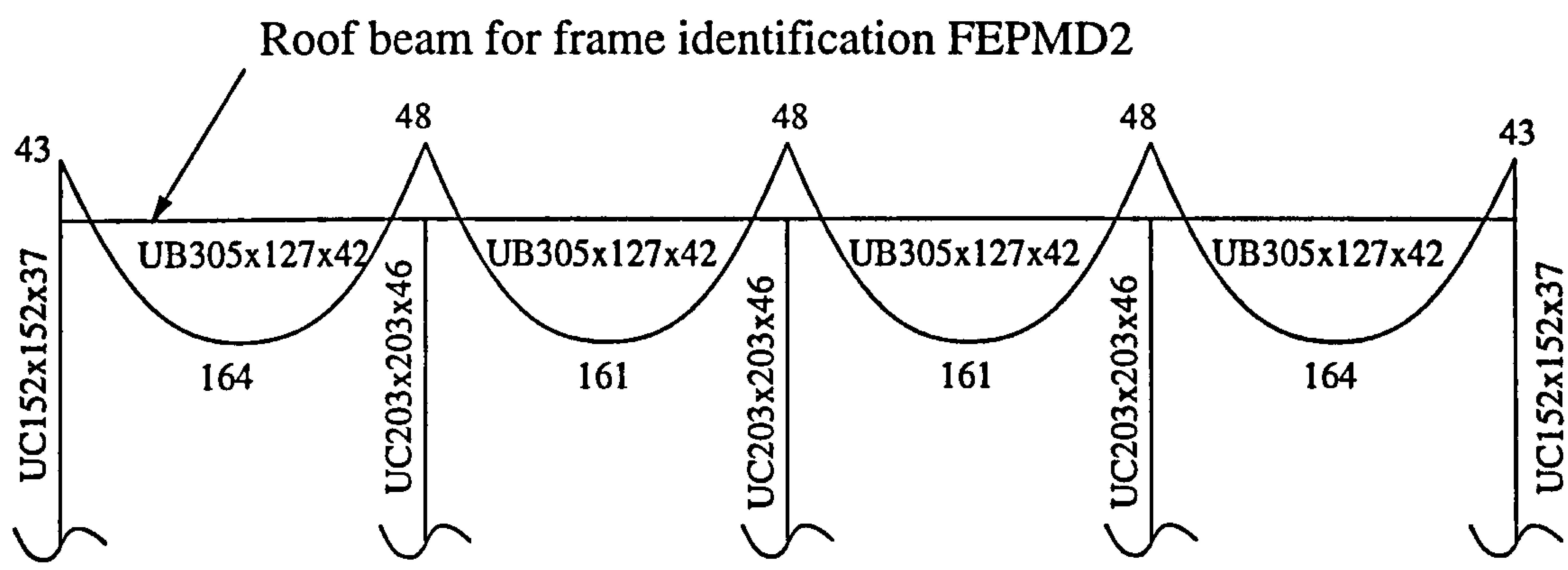


Figure 2.21 Effect of designed moment due to difference in connections resistance

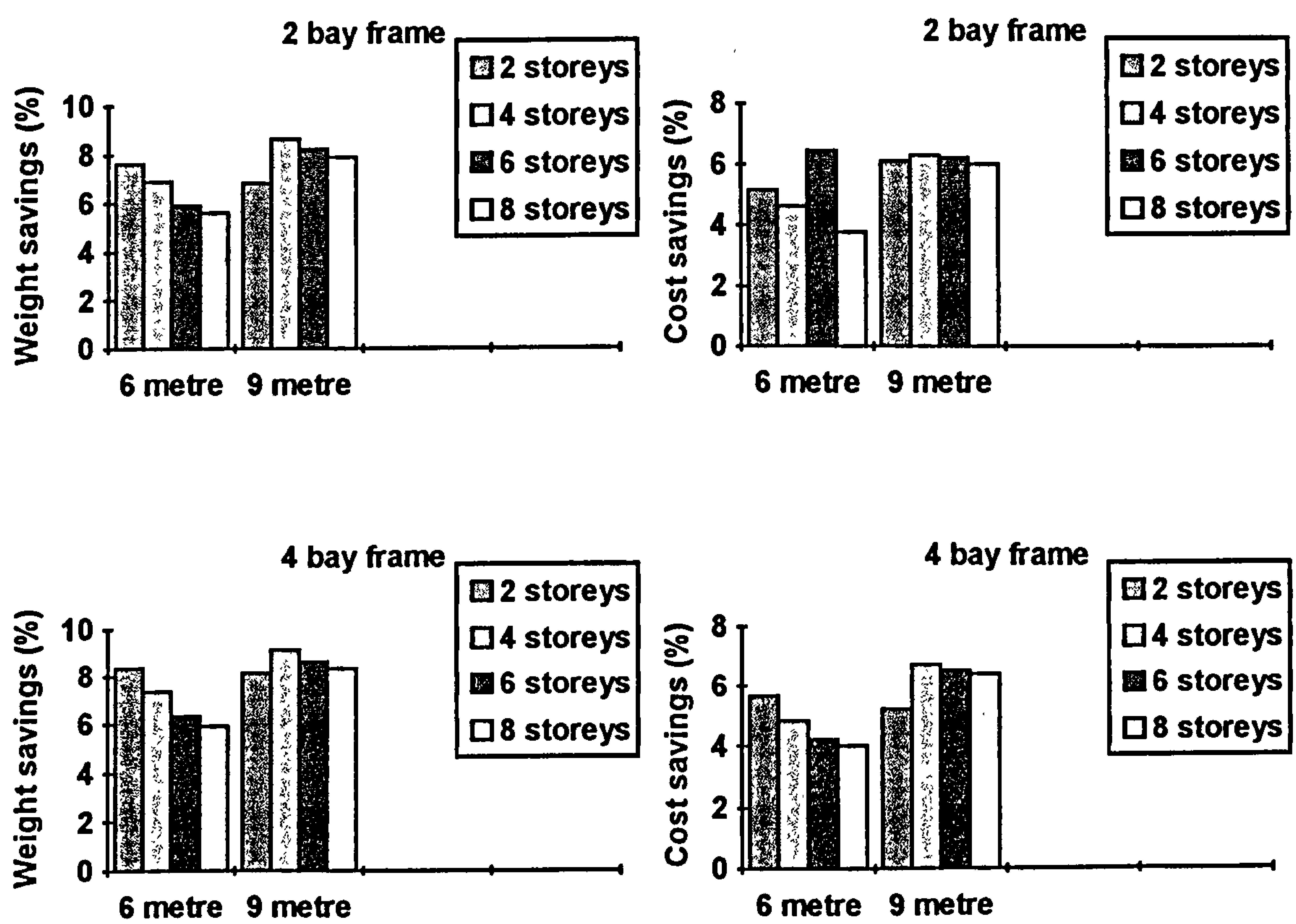


Figure 2.22 : Percentage weight and cost savings plotted against beam span

CHAPTER 3

3.0 Design of unbraced frames with bending about both column axes.

3.1 Introduction

Steel frames with bending about both the major and the minor axes of the column sections are usually designed on the basis that beam-to-column connections are either pinned or rigid. However, the actual behaviour will usually fall somewhere between these extremes, as recognised by the concept of semi-rigid design permitted by some design codes including Eurocode 3[3.1]. The connection behaviour is then represented by a moment-rotation (M-Φ) curve, relating the moment M transmitted by the connection to the relative rotation Φ between the beam and the adjacent column. This means that all connections, including connections connected to column web, will possess some moment capacity and some rotational stiffness. However, uncertainty concerning the behaviour of connections attached to a column web make this configuration an uncommon one for joints designed to be moment-resistant.

The use of the "wind-moment" design method in conjunction with BS 5950 Part 1[3.2] is now well-established for major-axis framing. This method effectively employs semi-rigid connections which are also "partial-strength" relative to the connected beams; a range of standard joints of this type are now available[3.3]. The existing rules for "wind-moment" design have been shown to provide adequate resistance to frames with major-axis sway. The "wind-moment" approach has been further extended by the present author to the design of unbraced frames with bending about both column axes, as described in this chapter. For frames which sway about the minor axis, the following are of particular concern:

1. the form of the minor-axis connection, which must provide reasonable moment resistance and stiffness;
2. the stiffness and stability of the frame against minor-axis sway, which will be influenced by the low flexural rigidity exhibited even by Universal Columns bent in this way;
3. in frames supporting precast units, the minor axis beams may remain as little more than tie members even when designed for wind moments, with consequent absence of appropriate stiffness to ensure frame stability at ultimate load and reasonable deflections in service.

The methodology is similar to that used previously on major-axis framing[3.4][3.5], namely the design of a series of frames by the proposed rules, followed by "exact" analysis[3.6] to check the designs. The "exact" analysis is described by Kavianpour as a second order elasto-plastic approach, based on the matrix displacement method. The analysis includes the behaviour of the connections by including an estimation of the secant stiffness of each connection obtained from the M-Φ characteristics. These in turn may be obtained from tests, mathematical expressions or analytical models[3.7]. Plasticity in members is taken into account by a plastic hinge idealization. The effect of axial force in frame stiffness is included by using stability functions with modifications made to the slope-deflection equations; small-deflection theory is assumed.

It has been assumed in the "exact" analyses that bases are fixed. BS5950 (clause 5.1.2.4) does not recognise such fixity, expect in the elastic-plastic design of sway frames given in clause 5.7.3.1. This exclusion provides justification for the author's assumption. Also, in view of the inclusion in the author's "exact" analyses of initial joint flexibility, plasticity in both members, and joints, and the neglect of "stray" composite action with floor and cladding elements[3.8], the neglect of the flexibility of nominally-fixed bases is regarded as reasonable.

3.2 Range of applications

The range of the study is summarised in Fig. 3.1. The frames ranged in height from two to eight storeys. In recognition of the unlikelihood of the frame consisting of only one longitudinal bay, the minimum number of bays in the minor axis framing was taken as two (see Fig 3.2). Each longitudinal bay was assumed to be 6m in length. The maximum number of longitudinal bays was taken in this study as six. The following configurations of minor-axis framing were therefore investigated:

- two-storey, two-bay
- four-storey, two-bay
- four-storey, four-bay
- four-storey, six-bay
- eight-storey, two-bay.

The limitations on frame dimensions conformed to those specified in the existing guide[3.9] for "wind-moment" design. In view of possible difficulty in ensuring adequate stability and stiffness, the study assumed S275 steel, rather than the higher grade material used in some of the earlier studies[3.5].

Two arrangements of floor grids were considered. The first consists of floor units assumed to span 6 m between the major-axis frames (see Figure 3.3(a)); this results in the minor-axis beams being free of significant gravity forces, the main loading being wind-moments. The second grid assumed composite floors spanning only 3m (see Figure 3.3(b)), with the result that the minor axis beams act as primary beams in support of the floor; substantial minor-axis beam sections are then needed to resist gravity forces and the inherent stability of the minor axis framing is significantly increased. The limitations on loading conformed to those in the existing recommendations[3.9]. Frames were designed for combinations of maximum gravity load with minimum wind forces, and vice-versa.

3.2.1 Load combinations

For serviceability limit states loads were taken as unfactored. When considering dead load plus imposed load and wind load only 80 % of the imposed load and wind load need to be considered[3.2]. Frames were be analysed under three load combinations as follows:-

- 1.0 Dead load plus 1.0 imposed load plus unfactored notional force
- 1.0 Dead load plus 0.8 imposed load plus 0.8 wind load
- 1.0 Dead load plus 1.0 wind load.

Deflection limits for a building with more than one storey are recommended by BS 5950 to be less than 1/300th of the height of the storey under consideration.

For ultimate limit states, loads were be taken as factored. Frames were analysed under three load combinations as follows:-

- 1.4 Dead load plus 1.6 imposed load plus factored notional force
- 1.2 Dead load plus 1.2 imposed load plus 1.2 wind load
- 1.4 Dead load plus 1.4 wind load.

3.2.2 Determination of wind forces and notional forces

3.2.2.1 Wind forces

Basic wind speeds were taken as the three-second gust speed estimated to be exceeded on average once in 50 years. Wind forces were calculated in accordance with CP3: Chapter V: Part 2[3.10], the code in use in practice at the time of the study. The formula for design wind speed V_s is given in CP 3 clause 4.3 as:

$$V_s = V\, S_1\, S_2\, S_3$$

where

V is the basic wind speed,

V_s is the design wind speed,

S_1 = topography factor (taken as 1.0),

S_2 = factor to account for ground roughness, building size and the height above ground,

S_3 = factor based on statistical concepts given in Appendix C of CP3.

The design wind speed is converted to dynamic pressure q using the relationship

$$q = k\, V_s^2.$$

The wind force on a surface is then given by : $F = (C_{pe} - C_{pi})qA$

where

C_{pe} is the pressure coefficient for the external face,

C_{pi} is the pressure coefficient for the internal face,

A is the area of the surface.

However, as the overall effect of the wind was required, rather than the local pressure on an individual surface, C_{pi} was not relevent. The total wind force was taken as $F = \left(\sum C_{pe}\right)qA$, where the summation is over the windward and leeward faces. Wind forces were considered as horizontal point loads acting on the windward external columns at each floor level. In design, account was taken of the compressive axial forces in the leeward columns, contributed by the horizontal wind. No account was taken of wind uplift on the roof, as this would relieve the compressive axial forces in the columns.

3.2.2.2 Notional forces

To ensure sway stability under gravity load, notional horizontal forces should be applied[3.2]. These notional forces were taken as the greater of:

- 1 % of the factored dead load from that level, applied horizontally
- 0.5 % of the factored load (dead plus vertical imposed) from that level, applied horizontally.

In practice, the latter governs in this study.

3.3 Design methodology

Initially the structure was designed by the wind-moment method assuming bending about the major axis of the columns. Computer software written by Reading[3.4] and modified by Brown[3.5] was used to design the column sections for frame bending about this axis. The modification by Brown was to change the effective length for minor axis buckling from 0.85L to 1.0L, in accordance with the published rules for wind-moment design[3.9]. For minor axis design, the software was further modified by the present author. The rules to proportion the individual members are described in Section 3.3.1 amd 3.3.2.

With floor grids arranged as in Figure 3.3(a), the minor axis beams are designed to resist only those moments due to either notional horizontal forces or wind. Small beam sections may then result. Second-order "exact" analyses[3.6] then show inadequate sway stability for the ULS design loads. Two procedures are adopted to stiffen the frame:

1. sections are increased to limit the sway index to 1/300 under serviceability wind forces;
2. further increases may be made in beam sections to provide improved restraint to the columns.

These procedures are described below in Section 3.3.2.1 and 3.3.2.2. For floor grids arranged as in Figure 3.3(b) where minor axis beams act as primary beams and therefore carry substantial gravity loads, their designs are discussed in Section 3.3.3. The form of connections proposed is presented in Section 3.3.4.

3.3.1 Wind-moment design for beams in minor-axis framing

For floor grids as in Figure 3.3(a), the beams are considered as tie beams which carry no gravity load except their own dead weight. This is because the floor units span directly between the major-axis framing and the minor axis beams may be positioned clear of the underside of the floor. In such circumstances the minor-axis beams resist bending moments M only due to the notional horizontal forces or due to wind. By being clear of the underside of the floor, the minor axis beam is laterally unrestrained. The member should therefore be designed as follows:

1. As the notional horizontal forces are a device to allow for initial out-of-plumb (and are therefore not real forces), the appropriate maximum slenderness is that corresponding to a member carrying only wind forces in addition to its own self weight. Hence, from clause 4.7.3.2 of BS5950 Part 1[3.2], the effective slenderness ratio should not exceed 250; in view of the end restraint, the slenderness ratio is taken as $0.85L/r_y$;
2. $M_{cx} \geq M$, where M_{cx} is the moment capacity about major axis. For a compact section, $M_{cx} = p_y S$ but $\leq 1.2 p_y Z$ where for semi-compact section, $M_{cx} = p_y Z$ where p_y is the design strength, S is the plastic modulus of the section about the relevent axis, and Z is the corresponding elastic modulus;
3. the moments due to notional horizontal forces and wind cause double curvature in the minor-axis beams as shown in Fig.3.4. The need to check for lateral torsional buckling under this moment distribution is discussed in Section 3.3.1.2.

For floor grids as in Figure 3.3(b), the design of the beams is in accordance with the recommendations for major-axis framing[3.9].

3.3.1.1 Check for maximum slenderness

Tie beams transmit wind forces and notional horizontal forces to distribute these between the bays. Hence they act as horizontal struts. According to BS 5950 clause 4.7.3.2, for members resisting self weight and wind loads only the maximum slenderness should not exceed 250. In this study, the minimum beam section for tie-beams was therefore a 203x133x25UB. The resulting slenderness ratio (λ) is equal to 165 based on $0.85L/r_y$ and a 6 metre span. However, the maximum slenderness ratio is equal to 238 calculated from 254x102x25UB which still not exceeding 250.

3.3.1.2 Check for lateral torsional buckling.

Since tie beams do not carry gravity load, they are considered as laterally unrestrained. In this case, *m* is taken to be equal to 0.43 due to the double curvature effect. The studies have shown though that lateral torsional bucking is not critical. Calculations are shown in Table 3.1(a to e) for minimum wind combined with maximum gravity load and in Table 3.2(a to e) for maximum wind combined with maximum gravity load. The beams shown are those selected after frames with precast floor have been designed to limiting sway as described in Section 3.3.2.1. Where different beams employ the same section in the same frame, that with the maximum end moment has been selected for the calculation.

3.3.2 Wind-moment design for columns in minor axis framing

For floor grids as in Figure 3.3(a), it is likely that the worst situation for frame stability will arise with the structure fully-loaded. For "internal" minor-axis framing (see Figure

3.5), with equal bay widths and loading, the only bending moments in the columns are due to the horizontal loads, and it may therefore be expected that the column moments will be in double-curvature bending. In the design of minor-axis framing for this study, an equivalent uniform moment factor m_y of 0.43 was therefore adopted for the overall buckling check (Clause 4.8.3.3.1 of BS 5950). In view of the earlier design recommendations for major-axis framing[3.9], it is proposed that m_x be taken as unity. For calculation of the buckling resistance moment the effective length is taken as 1.0 L. The effective length of the column for resistance to axial load is influenced by sway about the minor axis and should be taken as 1.5 L.

Patterned loading should be considered even though as just stated, it is unlikely to be critical in some cases. Patterned loading induces out-of-balance moments in the columns, which should therefore then be checked for biaxial bending. Patterned loading was developed by loading the beams on one side of the column, with the factored maximum gravity load (1.4 Dead load + 1.6 Imposed load) and the other side with the factored dead load only (1.4 Dead load). For floor grids as in Figure 3.3(a), the unbalanced moment acted about the major axis of the columns whereas wind or notional forces contributed moment about the minor axis. For floor grids as in Figure 3.3(b), a second pattern, to induce the maximum out-of-balance about the minor axis, was also considered. Figure 3.6 shows the two arrangements of loading for such grids.

3.3.2.1 Design to limiting sway deflections

As a result of its simplicity, the "wind-moment" approach is attractive to those who wish to continue to design by manual calculation. For rigid-jointed unbraced frames, hand methods are available to determine sway deflection[3.11]. One such method, due to Anderson and Islam[3.12] is outlined in Chapter 1, generates designs to specified limits on inter-storey sway. An element of optimisation is included, which permits account to be taken of the differing efficiencies of various section shapes in providing flexural rigidity. This method has been used in designing stiffer minor-axis framing, those sections already chosen by the "wind-moment" approach being taken as lower bounds on sizes.

Comparison of the more efficient Universal Beams in major-axis bending with the minor axis properties of Universal Columns (see Fig. 3.7) showed that the former are approximately five times more efficient in providing flexural rigidity. Account was taken of this difference when using the formulae by Anderson and Islam. Their factor k_3 accounts for such differences; the value taken was 4.8, as shown in Fig. 3.7. The effect of this factor is to encourage the use of deeper beams to provide overall sway stiffness.

Although the formulae apply to rigid-jointed frames, the use of deep beams necessarily leads to stiffer connections from the standard range[3.3], which encourages stability in the final design even when account is taken of joint flexibility. If however the formulae predicted that the optimum design required smaller columns than the "wind-moment" calculations allowed, the formulae were then used in an alternative mode. This enabled beam sections to be selected to meet the deflection limit, taking account of the rigidity of the already-chosen columns. To avoid an undue number of splices, column sections were only changed every two storeys for two and four storey frames, and two or three storeys for eight storeys frames.

For frames with the grid of Figure 3.3(a), the formulae were used in conjunction with a deflection limit of height/300 and the full unfactored wind load. In later discussions, the resulting design is denoted as 'Section Designation II'. The formulae are based on an assumed first-order elastic response. In the interests of research, the resulting designs were also subjected to computer analysis[3.13]. This was partly to check that the formulae had generated reasonably stiff designs, but it also permitted account to be taken of second-order effects. When these caused the limiting index of 1/300 to be exceeded, beam sections were further increased until the second-order (but still rigid) analysis showed this limit had been satisfied. The resulting design for the frame with a precast floor is denoted 'Section Designation III'.

For frames with the grid of Figure 3.3(b), the same formulae were used but in conjunction with a deflection limit of height/300 and height/450 analysed separately. The former is denoted as 'Section Designation II' whereas the latter is denoted as 'Section Designation III'. By limiting the deflection index to 1/300, all frames with rigid joints showed results not exceeding the limiting value for both first and second order analysis expect for 2 bay 8 storey frame identified as Frame 3 in Table 3.11 analysed for second order analysis.

However, when frames designed to a sway index of 1/300 were analysed with semi-rigid, the results showed that most of the frames exceeded this limit, especially in 2 bay 8 storey frames, and for others subject to maximum wind combined with minimum gravity load. Therefore, the formulae are used again with the limiting sway index increased to H/450 to redesign the frames ('Section Designation III'). With this increase, it is hoped that the sway deflection accounting for joint flexibility will not exceed the deflection index of 1/300.

3.3.2.2 Restraint to the columns

Despite the above procedure, it was found that designs for frames of four or more longitudinal bays, subject to minimum wind load combined with maximum gravity forces, still did not possess adequate overall stability for ULS design loads when account was taken of the M-Φ response of the semi-rigid and partial strength connections.. This is due to the following reasons:

- Minimum wind distributed amongst several bays. This leads to a low bending moment required for the design of the connection to an individual beam. As a result of the link between moment capacity, depth of connection and stiffness inherent in the standard connections, a flexible connection is chosen. The overall stiffness of the frame is thereby reduced.
- The gravity load contributes to the compressive axial forces in the column, which is the sources of the instability. As the gravity load increases in the maximum load case without a corresponding increase in sway stiffness, the overall stability of the structure

is reduced. This results in a low critical load factor λ_{cr} against side-sway and the frame may reach a point of collapse[3.14].

One possible solution would be to take account of connection flexibility in the Anderson and Islam formulae, by treating the required beam rigidity as an effective value; taking account of the connection flexibility would lead to a stiffer beam section then being selected[3.15]. However, this requires knowledge of connection stiffness. Inspection of the inadequate designs showed that they were characterised by beams in the range 203-305UB, restraining columns up to 356UC in size. It is therefore proposed that the following "rule-of-thumb" recommendations be adopted to ensure that the restraining beams have reasonable stiffness. With reference to Figure 3.8:

At 1: $\dfrac{(I_{bx})_1}{L} \geq \dfrac{(I_{cy})_1}{h_1}$

At 2: $\dfrac{(2I_{bx})_1}{L} \geq \dfrac{(I_{cy})_2}{h_1}$

At 3: $\dfrac{(I_{bx})_2}{L} \geq \dfrac{(I_{cy})_3}{h_2} + \dfrac{(I_{cy})_4}{h_3}$

At 4: $\dfrac{(2I_{bx})_2}{L} \geq \dfrac{(I_{cy})_5}{h_2} + \dfrac{(I_{cy})_6}{h_3}$

The subscripts x and y refers to the major and minor axes of the beam section and the column section respectively. The resulting design is denoted 'Section Designation IV'.

3.3.3 Frame grids for composite floors

Although the above recommendations (see Section 3.3.2.1 and 3.3.2.2) are primarily intended to avoid inadequate sway stability that would otherwise arise with grids as in Figure 3.3(a), they can also be applied to frames with grids as in Figure 3.3(b). All frames must possess adequate stiffness under serviceability loading. The recommendations represent good practice based on engineering judgement and may, in any case, be automatically satisfied by beams from the grids concerned, because these members support substantial gravity loads.

3.3.4 Minor-axis connections

Moment-resisting minor axis connections, in which the beam's end-plate is bolted to the column web (see Fig. 3.9) have been investigated by, amongst others, Kim[3.16] and Gomes[3.17]. Moment resistance is limited by the column web, but this may be determined by yield-line analysis[3.17]. Uncertainty however surrounds the prediction of the stiffness of such connections; to the author's knowledge there is no accepted method to calculate this property. To enable the frame study to proceed, it was therefore decided to assume that connections of the form shown in Figure 3.10 would be used. It was further assumed that the thickness of the plate welded between the column flanges, and the restraint to this plate, would be such that its contribution to the flexibility of the connection could be ignored. Thus the stiffness of the connection could be determined from the equation derived by Brown[3.5] but with the terms corresponding to bending of the column flange and in-plane shear deformation of the column web being omitted as described in Chapter 1. The moment resistance of the connections used were taken from the tabulated standard joints[3.3].

3.3.4.1 Connection stiffness at Serviceability Limit State.

In the analysis of "wind-moment" frames it is important to choose the correct behaviour for the connection stiffness. At first sight, it appears that an application of the service gravity load, followed by wind as for the case of floor grid in Fig 3.3(b), could result in one connection (the leeward) being loaded along the plastic plateau. The anticipated sequence of events is illustrated in Figure 3.11. Because of the "partial strength" nature of "wind-moment" connections, it is assumed in Figure 3.11 that the service gravity load causes some plastic deformation. Application of wind load causes a decrease in the existing moment at the windward connection which unloads, but continues the plastic deformation at the leeward side. The joint at the leeward is designed for 1.4(wind moment) + 10%(1.4 dead load moment)[3.9]. In reality at SLS, joints will have to resist 1.0(wind moment) + 10%(1.0 dead load moment). Therefore, the ratio of unfactored load to factored load is equal to 0.707(i.e. 1/1.4) which is greater than the 0.67 load ratio regarded as valid for the initial secant stiffness, $K_{j,ini}$[3.1]. This results in the use of $K_{j,ini}/2$ for leeward connection[3.18]. For the windward connection, $K_{j,ini}$ is used because dead load moment opposes wind moment and the anticipated moment is therefore less than $0.67M_{j,Rd}$.

For the floor grid as shown in Fig 3.3(a) in which only wind load acts on the minor-axis beams, the anticipated M-Φ curve is as shown in Fig 3.12 for both windward and leeward connections. At SLS, the expected wind moment is equal to 0.707 $M_{j,Rd}$(i.e. 1/1.4) which is greater than $0.67M_{j,Rd}$. Therefore, $K_{j,ini}/2$ is used for both windward and leeward connections.

3.3.4.2 Connection stiffness at Ultimate Limit State

As the design loads for ULS are significantly greater than the loads ever expected in practice, it is inappropriate now to consider the unloading response at the leeward joint. This joint is assumed to continue to load, and a bi-linear M-Φ relationship has therefore been used in the analyses. The rotation at which the plastic plateau begins has been determined by assuming a secant stiffness equal to half the initial value, as recommended by revised Annex J[3.18] for global analysis. The windward joint, at which gravity load effects and wind load are in opposition, remains represented by its initial stiffness. However, for floor grid as shown in Fig 3.3(a) with wind load only, a secant stiffness equal to half the initial value is used for both windward and leeward joints as no gravity load affects the joint. The stiffnesses of connections used for SLS and ULS analyses are summarised in Fig 3.13.

3.4 Parametric study.

The frame arrangements studied and the dimensions and loading, are listed in Tables 3.3, 3.5, 3.7, and 3.9. Tables 3.3 and 3.5 concern minimum wind combined with maximum gravity load and Tables 3.7 and 3.9, the reverse. The wind-moment designs ('Section Designation I') are given in these tables. To improve stiffness and satisfy the deflection limits, frames are designed with the proposed rules, and are listed in Table 3.4, 3.6, 3.8, and 3.10. For Frames 9 and 10, additional recommendations given in 3.3.2.2 are required; the resultant design is 'Section Designation IV' in Table 3.6. 'Section Designation III' (explained in 3.3.2.1) is not relevant to the verification of the design methodology, but will be referred to in the assessment of the results. For frames with floor grid as in Fig 3.3(b), the need to design for 'Section Designation II' and 'Section Designation III' is explained in 3.3.2.1. The load-deflection (sway) behaviour for each of the frame up to the point of collapse was examined for second-order analysis at ULS. An example of load-deflection curves for Frame 1 (Table 3.4) is shown in Fig. 3.14(a-c).

3.4.1 Verification.

To justify the design recommendations, the frames were subject to second-order analysis accounting for the semi-rigid and partial-strength nature of the joints. Software[3.13] was used to carry out this analysis. Explanation of the values of secant stiffness used up to the design moment resistance of the joint have been described in 3.3.4 above. On the attainment of the connection's design moment of resistance, the joint is represented by a plastic hinge sustaining this level of moment.

Generally, when the overall sway deflections were calculated, both first-order and second-order values were obtained. The resistance moment of the column sections was taken as the plastic moment about the minor axis, reduced to take account of co-existent axial force, in accordance with the usual formulae[3.19] given in British tables for steel sections. It should be noted that because of the shape factor about the minor axis, the attainment of the plastic moment at the end of a column will be accompanied by plastic zones of significant length away from the theoretical plastic hinge. The computer analysis does not account for the loss of stiffness resulting from partially-plastic regions. This does not invalidate the conclusions from the study because subsequent checks were made on the local behaviour of each column length as now described.

To ensure local column stability, checks on overall buckling and local capacity were made in accordance with Clause 4.8.3.3.1 and 4.8.3.2(a) of BS 5950[3.2] summarised in Chapter 1. The moments and forces were those given by the analysis at the design load levels for ULS. As the results were regarded as "exact", equivalent uniform moment factors were calculated from the distribution of bending moments revealed in the columns. The resulting comparisons against unity are termed 'Stability Factors'. For the overall buckling check, the minor axis moment of resistance was taken as the yield moment p_yZ_y, not the plastic moment. For the local capacity check, the moment capacity was taken as the less of the minor axis plastic moment and $1.2\,p_yZ_y$. The latter limit restricts the length of any partially-plastic zones which, as previously mentioned, were not accounted for in the computer analysis. For comparative purposes, the designs were also analysed as if they were frames with rigid and full-strength connections.

3.4.2 Assessment of results.

The results are summarised in Tables 3.11-3.18. For Frames 13 and 16 (2 bay 8 storey), no designs could be produced due to the very high wind moments which exceeded the resistance of the strongest standard connections. Otherwise, where 'N/A' is shown, this indicates that:

- in the case of Section Designation I, the design possessed inadequate overall stability;
- in the case of Section Designations III and IV, the design recommendations corresponding to these designations were unnecessary to achieve a satisfactory design;
- in the case of deflections, the overall sway exceeded the index limit of 1/300.

It should be noted that for both ULS and SLS checks, the load cases referred to in the tables are described in 3.2.1.

Analysing as semi-rigid frames (Tables 3.13-3.14, 3.17-3.18), it can be seen that for grids arranged as Figure 3.3(b) (Frames 1, 2, 3, 7, 8, 11, 12, 17, 18), Section Designation II always shows adequate overall stability (it will be recalled that a load factor of unity corresponds to the design load levels for ULS; the load cases for ULS are those listed previously). The minimum factor is 1.02, for an eight-storey two-bay frame (Frames 3, Table 3.13). Under service loading though, this frame deflects excessively, due to joint flexibility not accounted for in developing 'Section Designation II' designs. A final design would therefore be stiffer and stronger; for example, Section Designation III gives a minimum factor of 1.23. Amongst the other frames (Frame 11 and 12, Table 3.17) and (Frame 17 and 18, Table 3.18), the load factors are greater than 1.0 but they all deflect excessively. It will be recalled from 3.3.2.1 that Section Designation III corresponds to frames designed to a sway index of 1/450 assuming rigid joints. However, some of these frames (Frame 11 and 12, Table 3.17) and (Frame 17, Table 3.18) still exceed the deflection index of 1/300 taking account of joint flexibility. For frames with acceptable sway at SLS, the minimum collapse load factor is 1.29 times the ULS design loads (Frame 8, Table 3.14).

Tables 3.13 and 3.17 also show that for grids arranged as Figure 3.3(a) (Frames 4 to 6 and 14 to 16), Section Designation II again provides adequate resistance to ULS design loads, treating the frames as semi-rigid. These frames are only of two-bays though. For minor axis framing extending to four or six bays, minimum wind loading results in inadequate performance (Frames 9 and 10, Table 3.14), unless the additional rules of Section 3.3.2.2 above are applied (resulting in Section Designation IV). For designs to withstand maximum wind loading, Section Designation II remain inadequate on grounds of serviceability, as shown by Frames 19 and 20 in Table 3.18.

The spread of plasticity in members and connections for the final designs are shown in Appendix of this thesis. For the semi-rigid analyses, the load levels shown for individual hinges are those at which the analysis detected the hinges to be present; because the software only prints results at the ULS design load level and subsequently when a plastic hinge forms in a member, partial-strength connections may have first reached their limiting moment resistance at lower load levels than those recorded. It should be emphasised that the corresponding reduction in frame stiffness is, of course, accounted for as soon as it occurs.

Tables 3.19-3.36 recorded the "stability factors" for critical column lengths at the ULS design load levels. Tables 3.19-3.36 indicate that the buckling resistance of the individual column lengths was adequate. The simplified local capacity check of Clause 4.8.3.2(a) of BS 5950 is not satisfied in Frames 4-6, 14, 19, 20. However, Clause 4.8.3.2(b) permits a fully-plastic approach which, for columns in uniaxial minor axis bending results in the same procedure as the computer software used to verify overall stability. The results shown in the Appendix show that for the frames in question, the plastic capacity of the columns was not exceeded. There is a remaining concern though. As a result of the high shape factor for UC sections bending about the minor axis, significant plastic zones located away from the idealised hinge position are expected before the hinge forms fully. These are not accounted for in the frame analysis. To ensure safety it is therefore described that the local capacity check based on the yield moment is observed. The check is not satisfied only by bottom storey columns. As bases are assumed fixed, these columns have much greater restraint at the lower end, compared to that provided by the members and connections attached to the upper end. More moment is attracted at the lower end than is allowed for by a point of contraflexture at mid-height. A more realistic assumption for the point of contraflexure, at 2/3 of the distance from the base is proposed in Chapter 7. This would result in a greater wind-moment design value, to more closely match that given by the "exact" analysis.

Comparison of overall sway deflections for rigid and semi-rigid analyses, can be obtained from Tables 3.11-3.12 and 3.15-3.16 (rigid analyses) and Tables 3.13-3.14 and 3.17-3.18 (semi-rigid). For frames whose rigid deflections are greater than height/500, it can be seen that the multiplier of 1.5 previously used for major-axis framing[3.9] continues to be a reasonable allowance in most structures for connection flexibility. For Frame 17 though (Table 3.16 and 3.18), a multiplier of 2.0 would be more appropriate.

3.5 Conclusions.

Despite the assumption of relatively stiff minor-axis connections in which the sole source of flexibility is associated with the beam end-plate, a straightforward extension of the previous rules for wind moment design[3.9] does not always result in frames of adequate overall stability. This is particularly true of frames in which floor units span between major-axis beams. In this case the minor-axis beams, not being heavily loaded, may be of small section size and therefore too flexible to ensure overall frame stability. In addition, the neglect of second-order effects results in the likelihood that the moment resistance of the joints will be reached below the design load level, causing a major deterioration of stiffness.

Further design rules have been developed by recognising the need to limit sway under service loading. However, for minor axis framing which extends over several bays, even these rules do not ensure adequate ultimate stability if the wind forces are low. Additional rules, relating to the minimum beam stiffness to the stiffness of the columns, have been proposed The resulting designs examined so far have adequate stability.

In the second case, when flooring consists of composite slabs, the minor axis beams will necessarily resist significant gravity load. This results in increased section sizes for those members and the wind-moment designs are therefore much more stable. Even so, it will be necessary in some cases to further increase section sizes, to avoid excessive sway under service load. On the basis of limited results, a multiplier of 1.5 to correct for joint flexibility is reasonable for most frames.

In view in the scope of the studies, and the problems they reveal in providing a frame of adequate resistance, it is concluded that the use of the wind-moment method "in two directions" should be restricted to low rise frames not more than four storeys. The author has more confidence in the use of the method for frames having secondary beams (in the minor-axis direction) of a reasonable size and stiffness. Its use with frames whose minor-axis beams are little more than tie members (Fig. 3.3(a)) relies on a series of rules to ensure adequate stability. In frames such as these it is more appropriate to base design on an "exact" second-order analysis, rather than to rely on the rules described earlier. These features ensure that sway deflection remains within acceptable limits, and therefore do not cause large second-order moments in the columns.

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Table 3.1 Check for lateral torsional buckling.

Table 3.1(a): 2 bay 2 storey with minimum wind combined with maximum gravity load

Beam section	u	v	$\lambda = L_E/r_y$	$\lambda_{LT} = nuv\lambda$	$M_b = S_x p_b$ (kNm)	$\overline{M} = mM_A$ (kNm)
203x133x25	0.876	0.75	165	108.1	29.4	0.43x24=10.3

Table 3.1(b): 2 bay 4 storey with minimum wind combined with maximum gravity load

Beam section	u	v	$\lambda = L_E/r_y$	$\lambda_{LT} = nuv\lambda$	$M_b = S_x p_b$ (kNm)	$\overline{M} = mM_A$ (kNm)
203 x 133 x 25	0.876	0.75	165	108.1	29.4	0.43x11=4.7
356 x 127 x 33	0.864	0.83	197	141.3	39.6	0.43x54=23.2
406 x 140 x 39	0.859	0.87	176	131.5	59.8	0.43x79=34.0

Table 3.1(c): 2 bay 8 storey with minimum wind combined with maximum gravity load

Beam section	u	v	$\lambda = L_E/r_y$	$\lambda_{LT} = nuv\lambda$	$M_b = S_x p_b$ (kNm)	$\overline{M} = mM_A$ (kNm)
203x133x25	0.876	0.75	165	108.1	29.4	0.43x16=7.0
356x127x39	0.872	0.79	190	131.0	54.2	0.43x51=21.9
406 x 140 x 46	0.870	0.85	169	125.0	80.0	0.43x87=37.4
457 x 191 x 67	0.873	0.90	124	97.3	191.2	0.43x119=51.2
533x 210 x 82	0.865	0.92	116	92.3	286	0.43x193=83.0
533 x 210 x 92	0.872	0.91	113	89.7	345.4	0.43x232=99.8

Table 3.1(d): 4 bay 4 storey with minimum wind combined with maximum gravity load

Beam section	u	v	$\lambda = L_E/r_y$	$\lambda_{LT} = nuv\lambda$	$M_b = S_x p_b$ (kNm)	$\overline{M} = mM_A$ (kNm)
203x133x25	0.876	0.75	165	108.1	29.4	0.43x16=6.9
254x102x25	0.864	0.72	238	148.2	21.1	0.43x27=11.6
305x102x33	0.866	0.72	237	148.0	32.6	0.43x39=16.8

Table 3.1(e): 6 bay 4 storey with minimum wind combined with maximum gravity load

Beam section	u	v	$\lambda = L_E/r_y$	$\lambda_{LT} = nuv\lambda$ $n = 1$	$M_b = S_x p_b$ (kNm)	$\overline{M} = mM_A$ (kNm)
203x133x25	0.876	0.75	165	108.1	29.4	0.43x18=7.7
254x102x25	0.864	0.72	238	148.2	21.1	0.43x26=11.2

Table 3.2 Check for lateral torsional buckling.

Table 3.2(a): 2 bay 2 storey with maximum wind combined with minimum gravity load

Beam section	u	v	$\lambda = L_E/r_y$	$\lambda_{LT} = nuv\lambda$	$M_b = S_x p_b$ (kNm)	$\overline{M} = mM_A$ (kNm)
305 x 102 x 33	0.866	0.72	238	148	32.6	0.43x30=12.9
406 x 140 x 46	0.870	0.85	169	125	80.0	0.43x107=46.0

Table 3.2(b): 2 bay 4 storey with maximum wind combined with minimum gravity load

Beam section	u	v	$\lambda = L_E/r_y$	$\lambda_{LT} = nuv\lambda$	$M_b = S_x p_b$ (kNm)	$\overline{M} = mM_A$ (kNm)
305 x 102 x 33	0.866	0.72	237	148	32.6	0.43x38=16.3
457 x 152 x 74	0.870	0.80	156	109	186.5	0.43x120=51.6
533 x 210 x 92	0.872	0.91	113	90	345.4	0.43x207=89.0
610 x 229 x 101	0.863	0.93	107	86	438.1	0.43x318=136.7

Table 3.2(c): 2 bay 8 storey with maximum wind combined with minimum gravity load

Beam section	u	v	$\lambda = L_E/r_y$	$\lambda_{LT} = nuv\lambda$ $n = 1$	$M_b = S_x p_b$ (kNm)	$\overline{M} = mM_A$ (kNm)
305 x 102 x 33	0.866	0.72	237	148	32.6	0.43x43=18.5
457 x 191 x 74	0.876	0.88	122	94	225.4	0.43x138=59.3
533 x 210 x 92	0.872	0.91	113	90	345.4	0.43x240=103.2
610 x 229 x 125	0.873	0.91	103	90	529.5	0.43x436=187.5
686 x 254 x 125	0.862	0.95	97	79	651.1	0.43x528=227.0
762 x 267 x 147	0.857	0.95	95	77	879.6	0.43x615=264.5
838 x 292 x 176	0.856	0.96	86	71	1423	0.43x767=329.8

Table 3.2(d): 4 bay 4 storey with maximum wind combined with minimum gravity load

Beam section	u	v	$\lambda = L_E/r_y$	$\lambda_{LT} = nuv\lambda$	$M_b = S_x p_b$ (kNm)	$\overline{M} = mM_A$ (kNm)
254 x 102 x 25	0.864	0.72	238	148	21.1	0.43x19=8.2
406 x 140 x 39	0.859	0.87	176	132	59.8	0.43x60=25.8
457 x 152 x 67	0.867	0.83	159	114	148	0.43x104=44.7
533x 210 x 82	0.865	0.92	116	92	286	0.43x159=68.4

Table 3.2(e): 6 bay 4 storey with maximum wind combined with minimum gravity load

Beam section	u	v	$\lambda = L_E/r_y$	$\lambda_{LT} = nuv\lambda$	$M_b = S_x p_b$ (kNm)	$\overline{M} = mM_A$ (kNm)
203 x 133 x 25	0.876	0.75	165	108	29.4	0.43x13=5.6
356 x 127 x 39	0.872	0.79	190	131	54.2	0.43x40=17.2
406 x 140 x 46	0.870	0.85	169	125	80.0	0.43x69=29.7
457 x 152 x 52	0.859	0.87	164	123	102	0.43x106=45.6

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Floor LL	Roof DL	Roof LL	Basic Wind Speed (m/s)	Ground Roughness Factor	Section Designation (I) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Connection Requirements Bending Moment (kNm) Floor	Bending Moment Roof	Shear Force (kN) Floor	Shear Force Roof
2 Storey 2 Bay	Frame 1	6.0 (composite floor at 3m span)											1st 533x210x82	356x171x45	Upto 2nd Storey	254x254x73	305x305x97	1st 61	27	1st 174	69
4 Storey 2 Bay	Frame 2												1st 533x210x82	356x171x45	Upto 2nd Storey	305x305x97	356x368x129	1st 108	29	1st 183	69
													2nd 533x210x82					2nd 86		2nd 177	
													3rd 533x210x82		2nd-4th Storey	254x254x73	254x254x89	3rd 69		3rd 174	
8 Storey 2 Bay	Frame 3												1st 533x210x82	356x171x45	Upto 3rd Storey	356x368x153	356x406x235	1st 239	34	1st 251	69
													2nd 533x210x82					2nd 206		2nd 222	
													3rd 533x210x82		3rd-6th Storey	305x305x118	356x368x153	3rd 191		3rd 196	
													4th 533x210x82					4th 167		4th 188	
													5th 533x210x82					5th 142		5th 182	
													6th 533x210x82		6th-8th Storey	254x254x73	254x254x89	6th 115		6th 177	
													7th 533x210x82					7th 84		7th 172	
			6.0	5.0	2	6.0	5.0	7.5	3.75	1.5	37	4									
2 Storey 2 Bay	Frame 4	6.0 (precast floor at 6m span)											1st 203x133x25	203x133x25	Upto 2nd Storey	203x203x71	203x203x71	1st 24	7	1st 8	2
4 Storey 2 Bay	Frame 5												1st 305x102x25	203x133x25	Upto 2nd Storey	305x305x97	356x368x129	1st 79	11	1st 26	4
													2nd 203x133x25					2nd 54		2nd 18	
													3rd 203x133x25		2nd-4th Storey	203x203x60	254x254x73	3rd 33		3rd 11	
8 Storey 2 Bay	Frame 6												1st 457x152x52	203x133x25	Upto 3rd Storey	356x368x153	356x406x235	1st 232	16	1st 77	5
													2nd 406x140x46					2nd 193		2nd 64	
													3rd 406x140x39					3rd 172		3rd 57	
													4th 406x140x39		3rd-6th Storey	305x305x97	356x368x153	4th 147		4th 49	
													5th 356x127x33					5th 119		5th 40	
													6th 305x102x28					6th 87		6th 29	
													7th 203x133x25		6th-8th Storey	203x203x60	254x254x89	7th 51		7th 17	

Table 3.3 Wind-moment design for 2 bay frames considering minimum wind in conjunction with maximum gravity load

Basic Frame Type	Frame Identification	Width of Bay (m)	Section Designation (II) Universal Beam Floor	Section Designation (II) Universal Beam Roof		Section Designation (II) Universal Columns External	Section Designation (II) Universal Columns Internal	Section Designation (III) Universal Beam Floor	Section Designation (III) Universal Beam Roof		Section Designation (III) Universal Columns External	Section Designation (III) Universal Columns Internal
2 Storey 2 Bay	Frame 1	6 composite floor	1st 533x210x82	356x171x45	Upto 2nd Storey	254x254x73	305x305x97	1st N/A	N/A	Upto 2nd Storey	N/A	N/A
4 Storey 2 Bay	Frame 2	6 composite floor	1st 533x210x82 2nd 533x210x82 3rd 533x210x82	356x171x45	Upto 2nd Storey	305x305x97	356x368x129	1st 2nd N/A 3rd	N/A	Upto 2nd Storey	N/A	N/A
					2nd-4th Storey	254x254x73	254x254x89			2nd-4th Storey	N/A	N/A
8 Storey 2 Bay	Frame 3	6 composite floor	1st 533x210x92 2nd 533x210x82 3rd 533x210x82 4th 533x210x82 5th 533x210x82 6th 533x210x82 7th 533x210x82	356x171x45	Upto 3rd Storey	356x368x153	356x406x235	1st 610x229x101 2nd 610x229x101 3rd 533x210x101 4th 533x210x92 5th 533x210x82 6th 533x210x82 7th 533x210x82	406x140x46	Upto 3rd Storey	356x368x177	356x406x340
					3rd-6th Storey	356x368x129	356x368x202			3rd-6th Storey	356x368x153	356x406x287
					6th-8th Storey	254x254x89	356x368x129			6th-8th Storey	254x254x89	356x368x129
2 Storey 2 Bay	Frame 4	6 precast floor	1st 203x133x25	203x133x25	Upto 2nd Storey	203x203x71	305x305x97	1st 254x102x25	203x133x25	Upto 2nd Storey	203x203x71	305x305x97
4 Storey 2 Bay	Frame 5	6 precast floor	1st 406x140x39 2nd 406x140x39 3rd 356x127x33	203x133x25	Upto 2nd Storey	305x305x97	356x368x129	1st 406x140x39 2nd 406x140x39 3rd 356x127x33	203x133x25	Upto 2nd Storey	305x305x97	356x368x129
					2nd-4th Storey	254x254x73	305x305x97			2nd-4th Storey	254x254x73	305x305x97
8 Storey 2 Bay	Frame 6	6 precast floor	1st 533x210x92 2nd 533x210x82 3rd 533x210x82 4th 533x210x82 5th 457x191x67 6th 406x140x46 7th 356x127x39	203x133x25	Upto 3rd Storey	356x368x153	356x406x235	1st 533x210x92 2nd 533x210x82 3rd 533x210x82 4th 533x210x82 5th 457x191x67 6th 406x140x46 7th 406x140x39	305x102x25	Upto 3rd Storey	356x368x153	356x406x235
					3rd-6th Storey	356x368x129	356x368x202			3rd-6th Storey	356x368x129	356x368x202
					6th-8th Storey	254x254x89	356x368x129			6th-8th Storey	254x254x89	356x368x129

Table 3.4 Minimum wind in conjunction with maximum gravity load

Section Designation Use	Composite floor	Precast floor
Section Designation II	Anderson and Islam formulae (H/300)	Anderson and Islam formulae (H/300)
Section Designation III	Anderson and Islam formulae (H/450)	Kavianpour software (2nd order analysis)

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m^2) Floor DL	Floor LL	Roof DL	Roof LL	Basic Wind Speed (m/s)	Ground Roughness Factor	Section Designation (I) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Connection Requirements Bending Moment (kNm) Floor	Bending Moment (kNm) Roof	Shear Force (kN) Floor	Shear Force (kN) Roof
4 Storey 4 Bay	Frame 7	6.0 (composite floor at 3m span)											1st 533x210x82 2nd 533x210x82 3rd 533x210x82	356x171x45	Upto 2nd Storey	305x305x97	356x368x129	1st 79 2nd 68 3rd 59	27	1st 183 2nd 177 3rd 174	69
															2nd-4th Storey	254x254x73	254x254x89				
4 Storey 6 Bay	Frame 8												1st 533x210x82 2nd 533x210x82 3rd 533x210x82	356x171x45	Upto 2nd Storey	305x305x118	356x368x129	1st 79 2nd 68 3rd 59	27	1st 183 2nd 177 3rd 174	69
															2nd-4th Storey	254x254x73	254x254x89				
			6.0	5.0	2	6.0	5.0	7.5	3.75	1.5	37	4									
4 Storey 4 Bay	Frame 9	6.0 (precast floor at 6m span)											1st 203x133x25 2nd 203x133x25 3rd 203x133x25	203x133x25	Upto 2nd Storey	305x305x97	356x368x129	1st 39 2nd 27 3rd 16	5	1st 13 2nd 9 3rd 5	2
															2nd-4th Storey	203x203x60	254x254x89				
4 Storey 6 Bay	Frame 10												1st 203x133x25 2nd 203x133x25 3rd 203x133x25	203x133x25	Upto 2nd Storey	305x305x97	356x368x129	1st 26 2nd 18 3rd 11	4	1st 9 2nd 6 3rd 4	1
															2nd-4th Storey	203x203x60	254x254x89				

Table 3.5 Wind-moment design for frames of 4 and 6 bays considering minimum wind in conjunction with maximum gravity load

Basic Frame Type	Frame Identification	Width of Bay (m)	Section Designation (II)					Section Designation (III)					Section Designation (IV)				
			Universal Beam			Universal Columns		Universal Beam			Universal Columns		Universal Beam			Universal Columns	
			Floor	Roof		External	Internal	Floor	Roof		External	Internal	Floor	Roof		External	Internal
4 Storey 4 Bay	Frame 7	6 composite beam	1st 533x210x82 2nd 533x210x82 3rd 533x210x82	356x171x45	Upto 2nd Storey	305x305x97	356x368x129	1st 2nd N/A 3rd	N/A	Upto 2nd Storey	N/A	N/A	1st 2nd N/A 3rd	N/A	Upto 2nd Storey	N/A	N/A
					2nd-4th Storey	254x254x73	254x254x89			2nd-4th Storey	N/A	N/A			2nd-4th Storey	N/A	N/A
4 Storey 6 Bay	Frame 8	6 composite beam	1st 533x210x82 2nd 533x210x82 3rd 533x210x82	356x171x45	Upto 2nd Storey	305x305x118	356x368x129	1st 2nd N/A 3rd	N/A	Upto 2nd Storey	N/A	N/A	1st 2nd N/A 3rd	N/A	Upto 2nd Storey	N/A	N/A
					2nd-4th Storey	254x254x73	254x254x89			2nd-4th Storey	N/A	N/A			2nd-4th Storey	N/A	N/A
4 Storey 4 Bay	Frame 9	6 Precast beam	1st 305x102x25 2nd 254x102x25 3rd 203x133x25	203x133x25	Upto 2nd Storey	305x305x97	356x368x129	1st 305x102x25 2nd 305x102x25 3rd 254x102x25	203x133x25	Upto 2nd Storey	305x305x97	356x368x129	1st 457x152x52 2nd 406x140x39 3rd 305x102x28	254x102x25	Upto 2nd Storey	305x305x97	356x368x129
					2nd-4th Storey	203x203x60	254x254x89			2nd-4th Storey	203x203x60	254x254x89			2nd-4th Storey	203x203x60	254x254x89
4 Storey 6 Bay	Frame 10	6 Precast beam	1st 254x102x25 2nd 203x133x25 3rd 203x133x25	203x133x25	Upto 2nd Storey	305x305x97	356x368x129	1st 254x102x25 2nd 203x133x25 3rd 203x133x25	203x133x25	Upto 2nd Storey	305x305x97	356x368x129	1st 457x152x52 2nd 406x140x39 3rd 305x102x28	254x102x25	Upto 2nd Storey	305x305x97	356x368x129
					2nd-4th Storey	203x203x60	254x254x89			2nd-4th	203x203x60	254x254x89			2nd-4th	203x203x60	254x254x89

Table 3.6 Minimum wind in conjunction with maximum gravity load

Section Designation Use	Composite Floor	Precast Floor
Section Designation (II)	Anderson and Islam formulae (H/300)	Anderson and Islam formulae (H/300)
Section Desigantion (III)	Anderson and Islam formulae (H/450)	Kavianpour software (2nd order analysis)
Section Designation (IV)	Not Available	Proposed rules

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Floor LL	Roof DL	Roof LL	Basic Wind Speed (m/s)	Ground Roughness Factor	Section Designation (I) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Connection Requirements Bending Moment (kNm) Floor	Bending Moment Roof	Shear Force (kN) Floor	Shear Force Roof
2 Storey 2 Bay	Frame 11												1st 457x152x60	356x171x45	Upto 2nd Storey	254x254x73	305x305x118	1st 120	45	1st 111	69
4 Storey 2 Bay	Frame 12	6.0 (composite floor at 3m span)											1st 457x191x67 2nd 457x152x60 3rd 457x152x60	356x171x45	Upto 2nd Storey 2nd-4th Storey	356x368x153 254x254x89	356x406x235 305x305x118	1st 331 2nd 221 3rd 134	52	1st 172 2nd 140 3rd 115	69
8 Storey 2 Bay	Frame 13												1st 610x229x113 2nd 610x229x101 3rd 533z210x92 4th 533x210x82 5th 457x191x67 6th 457x152x60 7th 457x152x60	356x171x45	Upto 3rd Storey 3rd-6th Storey 6th-8th Storey	356x406x287 356x368x177 254x254x89	356x406x551 356x406x287 356x368x129	1st 780 2nd 628 3rd 541 4th 449 5th 353 6th 254 7th 151	57	1st 316 2nd 257 3rd 232 4th 206 5th 178 6th 150 7th 120	69
			6.0	5.0	2	6.0	3.5	4.0	3.75	1.5	52	1									
2 Storey 2 Bay	Frame 14												1st 356x127x33	203x133x25	Upto 2nd Storey	254x254x73	305x305x118	1st 107	30	1st 36	10
4 Storey 2 Bay	Frame 15												1st 457x191x67 2nd 406x140x46 3rd 356x127x33	203x133x25	Upto 2nd Storey 2nd-4th Storey	356x368x153 254x254x73	356x406x235 305x305x118	1st 318 2nd 207 3rd 120	38	1st 106 2nd 69 3rd 40	13
8 Storey 2 Bay	Frame 16	6.0 (precast floor at 6m span)											1st 610x229x113 2nd 610x229x101 3rd 533x210x82 4th 533x210x82 5th 457x191x67 6th 457x152x52 7th 406x140x39	203x133x25	Upto 3rd Storey 3rd-6th Storey 6th-8th Storey	356x406x287 356x368x153 254x254x89	356x406x551 356x406x287 356x368x129	1st 767 2nd 615 3rd 528 4th 436 5th 340 6th 240 7th 138	43	1st 256 2nd 205 3rd 176 4th 145 5th 113 6th 80 7th 46	14

Table 3.7 Wind-moment design for 2 bay frames considering maximum wind in conjunction with minimum gravity load

Basic Frame Type	Frame Identification	Width of Bay (m)	Section Designation (II) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Section Designation (III) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal
2 Storey 2 Bay	Frame 11	6 composite floor	1st 457x152x60	356x171x45	Upto 2nd Storey	305x305x97	356x368x129	1st 457x152x67	356x171x45	Upto 2nd Storey	305x305x118	356x368x153
4 Storey 2 Bay	Frame 12	6 composite floor	1st 610x229x101 2nd 533x210x92 3rd 457x152x74	356x171x45	Upto 2nd Storey	356x368x153	356x406x287	1st 533x210x82 2nd 610x229x113 3rd 686x254x125	406x140x46	Upto 2nd Storey	356x368x153	356x406x287
					2nd-4th Storey	356x368x129	356x368x202			2nd-4th Storey	356x406x235	356x406x393
8 Storey 2 Bay	Frame 13	6 composite floor	1st 838x292x194 2nd 838x292x176 3rd 762x267x147 4th 610x229x125 5th 610x229x125 6th 533x210x92 7th 457x191x74	356x171x45	Upto 3rd Storey	356x406x340	356x406x551	1st 2nd 3rd 4th N/A 5th 6th 7th	N/A	Upto 3rd Storey	N/A	N/A
					3rd-6th Storey	356x406x235	356x406x393			3rd-6th Storey	N/A	N/A
					6th-8th Storey	356x368x129	356x406x235			6th-8th Storey	N/A	N/A
2 Storey 2 Bay	Frame 14	6 precast floor	1st 406x140x46	305x102x33	Upto 2nd Storey	305x305x97	356x368x129	1st 457x191x82	406x140x39	Upto 2nd Storey	305x305x97	356x368x129
4 Storey 2 Bay	Frame 15	6 precast floor	1st 610x229x101 2nd 533x210x92 3rd 457x152x74	305x102x33	Upto 2nd Storey	356x368x153	356x406x287	1st 610x229x113 2nd 610x229x101 3rd 457x191x74	356x127x33	Upto 2nd Storey	356x368x153	356x406x287
					2nd-4th Storey	356x368x129	356x368x202			2nd-4th Storey	356x368x129	356x368x202
8 Storey 2 Bay	Frame 16	6 precast floor	1st 838x292x176 2nd 838x292x176 3rd 762x267x147 4th 610x229x125 5th 610x229x125 6th 533x210x92 7th 457x191x74	305x102x33	Upto 3rd Storey	356x406x340	356x406x551	1st 838x292x194 2nd 838x292x176 3rd 762x267x147 4th 686x254x125 5th 686x254x125 6th 533x210x92 7th 533x210x82	406x140x46	Upto 3rd Storey	356x406x340	356x406x551
					3rd-6th Storey	356x406x235	356x406x393			3rd-6th Storey	356x406x235	356x406x393
					6th-8th Storey	356x368x129	356x406x235			6th-8th Storey	356x368x129	356x406x235

Table 3.8 Maximum wind in conjunction with minimum gravity load

Section Designation Use	Composite floor	Precast floor
Section Designation II Section Designation III	Anderson and Islam formulae (H/300) Anderson and Islam formulae (H/450)	Anderson and Islam formulae (H/300) Kavianpour software (2nd order analysis)

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column: Ground (m)	Height of Column: Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m^2): Floor DL	Gravity Load (kN/m^2): Floor LL	Gravity Load (kN/m^2): Roof DL	Gravity Load (kN/m^2): Roof LL	Basic Wind Speed (m/s)	Ground Roughness Factor	Section Designation (I): Universal Beam Floor	Section Designation (I): Universal Beam Roof	Section Designation (I)	Section Designation (I): Universal Columns External	Section Designation (I): Universal Columns Internal	Connection Requirements: Bending Moment (kNm) Floor	Connection Requirements: Bending Moment (kNm) Roof	Connection Requirements: Shear Force (kN) Floor	Connection Requirements: Shear Force (kN) Roof
4 Storey 4 Bay	Frame 17	6.0 (composite floor at 3m span)											1st 457x152x60 2nd 457x152x60 3rd 457x152x60	356x171x45	Upto 2nd Storey 2nd-4th Storey	305x305x97 203x203x60	356x368x153 254x254x73	1st 172 2nd 117 3rd 76	33	1st 126 2nd 111 3rd 104	69
4 Storey 6 Bay	Frame 18												1st 457x152x60 2nd 457x152x60 3rd 457x152x60	356x171x45	Upto 2nd Storey 2nd-4th Storey	305x305x118 254x254x73	356x368x129 254x254x89	1st 119 2nd 84 3rd 59	28	1st 104 2nd 105 3rd 111	69
			6.0	5.0	2	6.0	3.5	4.0	3.75	1.5	52	1									
4 Storey 4 Bay	Frame 19	6.0 (precast floor at 6m span)											1st 406x140x39 2nd 356x127x33 3rd 203x133x25	203x133x25	Upto 2nd Storey 2nd-4th Storey	305x305x97 203x203x52	356x368x153 254x254x73	1st 159 2nd 104 3rd 60	19	1st 53 2nd 35 3rd 20	6
4 Storey 6 Bay	Frame 20												1st 356x127x33 2nd 305x102x25 3rd 203x133x25	203x133x25	Upto 2nd Storey 2nd-4th Storey	254x254x89 203x203x46	305x305x118 254x254x73	1st 106 2nd 69 3rd 40	13	1st 35 2nd 23 3rd 13	4

Table 3.9 Wind-moment design for frames of 4 and 6 bays considering maximum wind in conjunction with minimum gravity load

Basic Frame Type	Frame Identification	Width of Bay (m)	Section Designation (II) Universal Beam Floor	Section Designation (II) Universal Beam Roof		Section Designation (II) Universal Columns External	Section Designation (II) Universal Columns Internal	Section Designation (III) Universal Beam Floor	Section Designation (III) Universal Beam Roof		Section Designation (III) Universal Columns External	Section Designation (III) Universal Columns Internal
4 Storey 4 Bay	Frame 17	6 composite beam	1st 533x210x82 2nd 457x152x67 3rd 457x152x60	356x171x45	Upto 2nd Storey	305x305x118	356x368x153	1st 533x210x92 2nd 533x210x82 3rd 457x152x67	356x171x45	Upto 2nd Storey	356x368x129	356x406x235
					2nd-4th Storey	305x305x97	356x368x129			2nd-4th Storey	305x305x118	356x368x153
4 Storey 6 Bay	Frame 18	6 composite beam	1st 457x152x60 2nd 457x152x60 3rd 457x152x60	356x171x45	Upto 2nd Storey	305x305x97	356x368x129	1st 533x210x82 2nd 457x152x67 3rd 457x152x60	356x171x45	Upto 2nd Storey	305x305x118	356x368x153
					2nd-4th Storey	254x254x73	305x305x118			2nd-4th Storey	305x305x97	356x368x129
4 Storey 4 Bay	Frame 19	6 Precast beam	1st 533x210x82 2nd 457x152x67 3rd 406x140x39	254x102x25	Upto 2nd Storey	305x305x118	356x368x153	1st 533x210x82 2nd 457x191x74 3rd 406x140x46	305x102x25	Upto 2nd Storey	305x305x118	356x368x153
					2nd-4th Storey	305x305x97	356x368x129			2nd-4th Storey	305x305x97	356x368x129
4 Storey 6 Bay	Frame 20	6 Precast beam	1st 457x152x52 2nd 406x140x46 3rd 356x127x39	203x133x25	Upto 2nd Storey	305x305x97	356x368x129	1st 457x152x60 2nd 406x140x46 3rd 406x140x39	356x127x33	Upto 2nd Storey	305x305x97	356x368x129
					2nd-4th Storey	254x254x73	305x305x118			2nd-4th	254x254x73	305x305x118

Table 3.10 Maximum wind in conjunction with minimum gravity load

Section Designation Use	Composite Floor	Precast Floor
Section Designation (II)	Anderson and Islam formulae (H/300)	Anderson and Islam formulae (H/300)
Section Designantion (III)	Anderson and Islam formulae (H/450)	Kavianpour software (2nd order analysis)

Basic Frame Type	Width of Bay (m)	Frame Identification	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)
2 Storey 2 Bay	6 composite floor	Frame 1	(I)	LC1	1.83	1/2245	(II)	LC1	1.83	1/2245	1/1930	(III)	LC1	N/A	N/A	N/A
				LC2	2.20	1/1014		LC2	2.20	1/1014	1/894		LC2			
				LC3	2.64	1/815		LC3	2.64	1/815	1/759		LC3			
4 Storey 2 Bay	6 composite floor	Frame 2	(I)	LC1	1.83	1/1743	(II)	LC1	1.83	1/1743	1/1469	(III)	LC1	N/A	N/A	N/A
				LC2	1.83	1/614		LC2	1.83	1/614	1/533		LC2			
				LC3	2.28	1/492		LC3	2.28	1/492	1/457		LC3			
8 Storey 2 Bay	6 composite floor	Frame 3	(I)	LC1	N/A	N/A	(II)	LC1	1.74	1/1459	1/1217	(III)	LC1	2.07	1/2193	1/1943
				LC2				LC2	1.53	1/397	1/340		LC2	1.97	1/603	1/547
				LC3				LC3	1.74	1/306	1/285		LC3	2.27	1/466	1/447
2 Storey 2 Bay	6 Precast floor	Frame 4	(I)	LC1	N/A	N/A	(II)	LC1	2.07	1/973	1/728	(III)	LC1	2.20	1/1100	1/846
				LC2				LC2	1.96	1/423	1/331		LC2	2.09	1/ 478	1/385
				LC3				LC3	2.35	1/338	1/296		LC3	2.53	1/382	1/339
4 Storey 2 Bay	6 Precast floor	Frame 5	(I)	LC1	N/A	N/A	(II)	LC1	1.85	1/1207	1/968	(III)	LC1	N/A	N/A	N/A
				LC2				LC2	1.77	1/421	1/350		LC2			
				LC3				LC3	2.02	1/337	1/307		LC3			
8 Storey 2 Bay	6 Precast floor	Frame 6	(I)	LC1	N/A	N/A	(II)	LC1	1.84	1/1496	1/1265	(III)	LC1	1.84	1/1541	1/1306
				LC2				LC2	1.60	1/400	1/348		LC2	1.60	1/404	1/352
				LC3				LC3	1.82	1/309	1/289		LC3	1.82	1/323	1/303

Table 3.11 ULS collapse load factor and deflection at SLS for rigid jointed 2 bay frames

Basic Frame Type	Width of Bay (m)	Frame Identification	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)
				LC1	1.83	1/1680	1/1419		LC1					LC1			
4 Storey 4 Bay	6 composite floor	Frame 7	(II)	LC2	2.16	1/1173	1/1019	(III)	LC2	N/A	N/A	N/A	(IV)	LC2	N/A	N/A	N/A
				LC3	2.64	1/942	1/875		LC3					LC3			
				LC1	1.83	1/2333	1/1926		LC1					LC1			
4 Storey 6 Bay	6 composite floor	Frame 8	(II)	LC2	2.21	1/2386	1/2061	(III)	LC2	N/A	N/A	N/A	(IV)	LC2	N/A	N/A	N/A
				LC3	2.65	1/1591	1/1479		LC3					LC3			
				LC1	1.38	1/623	1/395		LC1	1.46	1/693	1/459		LC1			
4 Storey 4 Bay	6 Precast floor	Frame 9	(II)	LC2	1.46	1/416	1/233	(III)	LC2	1.54	1/478	1/340	(IV)	LC2	N/A	N/A	N/A
				LC3	1.74	1/329	1/286		LC3	1.84	1/382	1/325		LC3			
				LC1	1.17	1/554	1/308		LC1					LC1			
4 Storey 6 Bay	6 Precast floor	Frame 10	(II)	LC2	1.43	1/580	1/358	(III)	LC2	N/A	N/A	N/A	(IV)	LC2	N/A	N/A	N/A
				LC3	1.85	1/455	1/365		LC3					LC3			

Table 3.12 ULS collapse load factor and deflection at SLS for 4 and 6 bay rigid jointed frames

Basic Frame Type	Width of Bay (m)	Frame Identification	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)
2 Storey 2 Bay	6 composite beam	Frame 1	(I)	LC1 LC2 LC3	1.31 1.59 1.89	1/840 1/537 1/512	(II)	LC1 LC2 LC3	1.31 1.59 1.89	1/840 1/537 1/512	1/733 1/474 1/476	(III)	LC1 LC2 LC3	N/A	N/A	N/A
4 Storey 2 Bay	6 composite beam	Frame 2	(I)	LC1 LC2 LC3	1.41 1.46 1.68	1/1250 1/476 1/400	(II)	LC1 LC2 LC3	1.41 1.46 1.68	1/1250 1/476 1/400	1/1019 1/398 1/370	(III)	LC1 LC2 LC3	N/A	N/A	N/A
8 Storey 2 Bay	6 composite beam	Frame 3	(I)	LC1 LC2 LC3	N/A	N/A	(II)	LC1 LC2 LC3	1.44 1.02 1.05	1/693 1/268 1/224	1/544 1/217 1/203	(III)	LC1 LC2 LC3	1.62 1.23 1.23	1/852 1/331 1/278	1/717 1/285 1/259
2 Storey 2 Bay	6 Precast Beam	Frame 4	(I)	LC1 LC2 LC3	N/A	N/A	(II)	LC1 LC2 LC3	1.23 1.16 1.37	N/A	N/A	(III)	LC1 LC2 LC3	1.47 1.31 1.56	N/A	N/A
4 Storey 2 Bay	6 Precast Beam	Frame 5	(I)	LC1 LC2 LC3	N/A	N/A	(II)	LC1 LC2 LC3	1.37 1.09 1.18	N/A	N/A	(III)	LC1 LC2 LC3	N/A	N/A	N/A
8 Storey 2 Bay	6 Precast Beam	Frame 6	(I)	LC1 LC2 LC3	N/A	N/A	(II)	LC1 LC2 LC3	1.65 1.03 1.02	N/A	N/A	(III)	LC1 LC2 LC3	1.70 1.05 1.03	N/A	N/A

Table 3.13 ULS collapse load factor and deflection at SLS for semi-rigid 2 bay frames

Basic Frame Type	Width of Bay (m)	Frame Identification	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check		Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check		Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check	
						(1st order)	(2nd order)				(1st order)	(2nd order)				(1st order)	(2nd order)
4 Storey 4 Bay	6 composite floor	Frame 7	(II)	LC1	1.31	1/605	1/487	(III)	LC1	N/A	N/A	N/A	(IV)	LC1	N/A	N/A	N/A
				LC2	1.53	1/511	1/423		LC2					LC2			
				LC3	1.89	1/515	1/467		LC3					LC3			
4 Storey 6 Bay	6 composite floor	Frame 8	(II)	LC1	1.29	1/636	1/512	(III)	LC1	N/A	N/A	N/A	(IV)	LC1	N/A	N/A	N/A
				LC2	1.59	1/686	1/572		LC2					LC2			
				LC3	1.90	1/727	1/660		LC3					LC3			
4 Storey 4 Bay	6 Precast floor	Frame 9	(II)	LC1	0.70	N/A	N/A	(III)	LC1	0.83	N/A	N/A	(IV)	LC1	1.37	1/693	1/471
				LC2	0.74				LC2	0.87				LC2	1.36	1/436	1/270
				LC3	0.93				LC3	1.07				LC3	1.58	1/367	1/310
4 Storey 6 Bay	6 Precast floor	Frame 10	(II)	LC1	0.56	N/A	N/A	(III)	LC1	0.56	N/A	N/A	(IV)	LC1	1.34	1/734	1/505
				LC2	0.68				LC2	0.68				LC2	1.65	1/727	1/530
				LC3	0.93				LC3	0.93				LC3	1.98	1/569	1/486

Table 3.14 ULS collapse load factor and deflection at SLS for semi-rigid 4 and 6 bay frames

Basic Frame Type	Width of Bay (m)	Frame Identification	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)
2 Storey 2 Bay	6 composite floor	Frame 11	(II)	LC1	2.00	1/4583	1/4400	(III)	LC1	2.03	1/5500	1/5189
				LC2	2.09	1/361	1/344		LC2	2.28	1/428	1/410
				LC3	2.04	1/290	1/281		LC3	2.42	1/343	1/333
4 Storey 2 Bay	6 composite floor	Frame 12	(II)	LC1	2.04	1/3500	1/3281	(III)	LC1	2.34	1/8750	1/8400
				LC2	2.01	1/376	1/362		LC2	2.73	1/571	1/557
				LC3	1.82	1/301	1/295		LC3	2.68	1/458	1/451
8 Storey 2 Bay	6 composite floor	Frame 13	(II)	LC1	2.04	1/6457	1/6212	(III)	LC1	N/A	N/A	N/A
				LC2	1.90	1/373	1/362		LC2			
				LC3	1.71	1/299	1/293		LC3			
2 Storey 2 Bay	6 Precast floor	Frame 14	(II)	LC1	5.66	1/4074	1/3793	(III)	LC1	5.82	1/5076	1/4803
				LC2	2.17	1/309	1/293		LC2	2.25	1/386	1/368
				LC3	2.02	1/248	1/239		LC3	2.08	1/309	1/300
4 Storey 2 Bay	6 Precast floor	Frame 15	(II)	LC1	6.57	1/5833	1/5526	(III)	LC1	6.59	1/6105	1/5833
				LC2	2.02	1/366	1/352		LC2	2.03	1/387	1/373
				LC3	1.83	1/292	1/286		LC3	1.84	1/309	1/301
8 Storey 2 Bay	6 Precast floor	Frame 16	(II)	LC1	6.48	1/6457	1/6212	(III)	LC1	6.49	1/6667	1/6436
				LC2	1.91	1/368	1/356		LC2	1.91	1/381	1/370
				LC3	1.72	1/294	1/289		LC3	1.72	1/305	1/300

Table 3.15 ULS collapse load factor and deflection at SLS for rigid jointed 2 bay frames

Basic Frame Type	Width of Bay (m)	Frame Identification	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)
4 Storey 4 Bay	6 composite beam	Frame 17	(II)	LC1	2.00	1/3134	1/2917	(III)	LC1	2.03	1/4565	1/4375
				LC2	1.82	1/410	1/380		LC2	2.36	1/598	1/571
				LC3	1.75	1/329	1/315		LC3	2.74	1/479	1/466
4 Storey 6 Bay	6 composite beam	Frame 18	(II)	LC1	1.83	1/3621	1/3182	(III)	LC1	2.00	1/4038	1/3684
				LC2	1.79	1/479	1/429		LC2	2.28	1/619	1/574
				LC3	1.84	1/369	1/347		LC3	2.33	1/485	1/464
4 Storey 4 Bay	6 Precast beam	Frame 19	(II)	LC1	3.43	1/2896	1/2658	(III)	LC1	3.43	1/3088	1/2838
				LC2	1.83	1/367	1/340		LC2	1.83	1/393	1/365
				LC3	1.75	1/294	1/281		LC3	1.75	1/314	1/301
4 Storey 6 Bay	6 Precast beam	Frame 20	(II)	LC1	2.68	1/2100	1/1842	(III)	LC1	2.70	1/2283	1/2019
				LC2	1.80	1/372	1/333		LC2	1.84	1/410	1/367
				LC3	1.85	1/297	1/278		LC3	1.87	1/326	1/305

Table 3.16 ULS collapse load factor and deflection at SLS for 4 and 6 bay rigid jointed frames

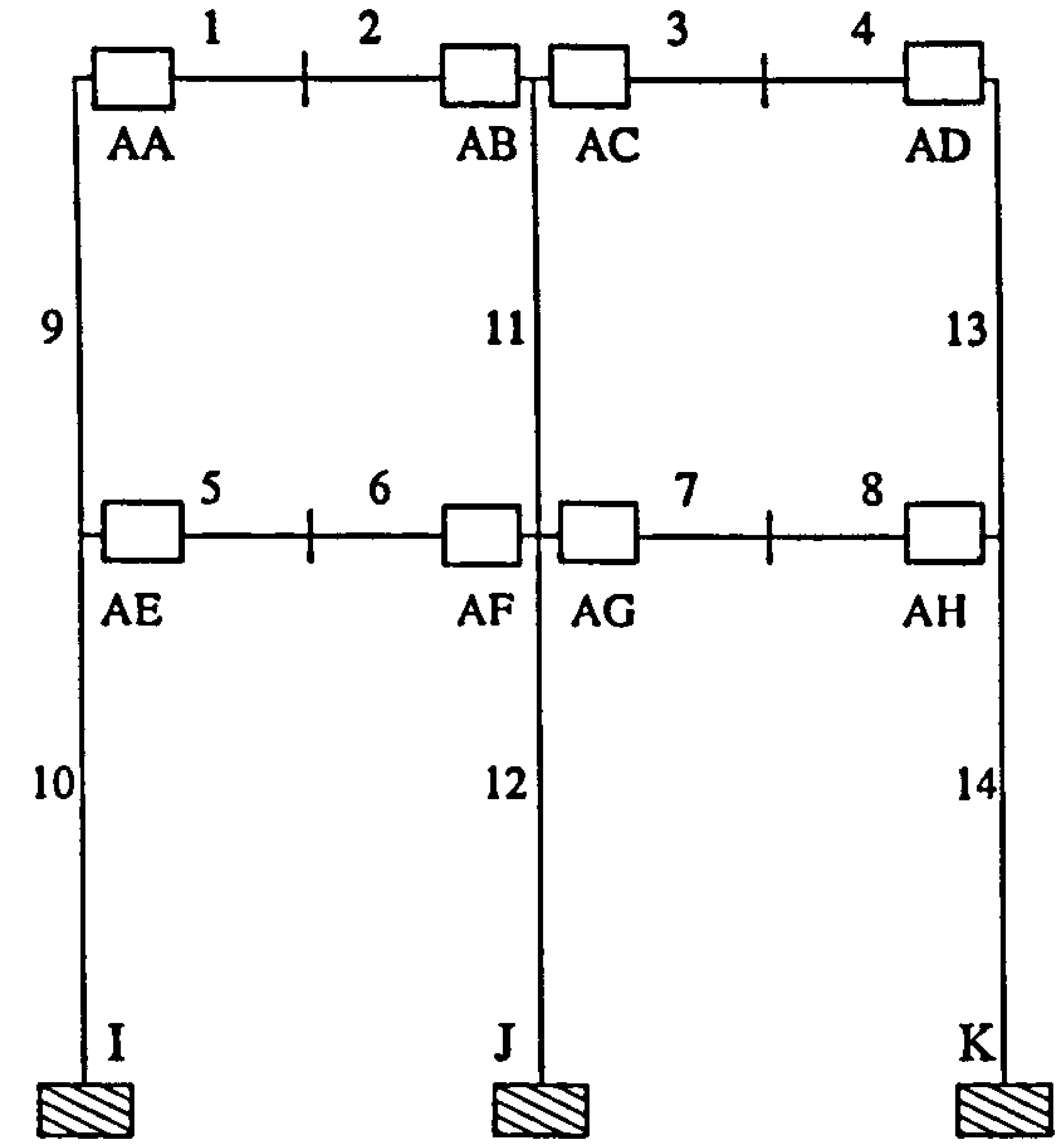
Basic Frame Type	Width of Bay (m)	Frame Identification	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)
2 Storey 2 Bay	6 composite beam	Frame 11	(II)	LC1	1.32	1/1429	1/1341	(III)	LC1	1.45	1/1507	1/1447
				LC2	1.37	1/248	1/234		LC2	1.48	1/279	1/265
				LC3	1.43	1/208	1/201		LC3	1.60	1/236	1/228
4 Storey 2 Bay	6 composite beam	Frame 12	(II)	LC1	1.45	1/1795	1/1707	(III)	LC1	1.67	1/2414	1/2333
				LC2	1.49	1/238	1/226		LC2	1.49	1/333	1/323
				LC3	1.39	1/197	1/191		LC3	1.45	1/277	1/272
8 Storey 2 Bay	6 composite beam	Frame 13	(II)	LC1	N/A	N/A	N/A	(III)	LC1	N/A	N/A	N/A
				LC2					LC2			
				LC3					LC3			
2 Storey 2 Bay	6 Precast Beam	Frame 14	(II)	LC1	4.85	N/A	N/A	(III)	LC1	5.43	N/A	N/A
				LC2	1.48				LC2	1.74		
				LC3	1.38				LC3	1.61		
4 Storey 2 Bay	6 Precast Beam	Frame 15	(II)	LC1	6.17	N/A	N/A	(III)	LC1	6.59	N/A	N/A
				LC2	1.36				LC2	2.03		
				LC3	1.21				LC3	1.84		
8 Storey 2 Bay	6 Precast Beam	Frame 16	(II)	LC1	N/A	N/A	N/A	(III)	LC1	N/A	N/A	N/A
				LC2					LC2			
				LC3					LC3			

Table 3.17 ULS collapse load factor and deflection at SLS for semi-rigid 2 bay frames

Basic Frame Type	Width of Bay (m)	Frame Identification	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)	Section Designation Use	Load Case	Collapse Load Factor (2nd order)	Deflection Check (1st order)	Deflection Check (2nd order)
4 Storey 4 Bay	6 composite beam	Frame 17	(II)	LC1 LC2 LC3	1.31 1.12 1.14	1/827 1/205 1/175	1/737 1/182 1/164	(III)	LC1 LC2 LC3	1.31 1.38 1.28	1/747 1/236 1/211	1/686 1/217 1/200
4 Storey 6 Bay	6 composite beam	Frame 18	(II)	LC1 LC2 LC3	1.43 1.15 1.11	1/1055 1/300 1/253	1/925 1/261 1/235	(III)	LC1 LC2 LC3	1.43 1.31 1.42	1/1193 1/370 1/318	1/1082 1/335 1/300
4 Storey 4 Bay	6 Precast beam	Frame 19	(II)	LC1 LC2 LC3	3.05 1.17 1.07	N/A	N/A	(III)	LC1 LC2 LC3	3.09 1.58 1.17	N/A	N/A
4 Storey 6 Bay	6 Precast beam	Frame 20	(II)	LC1 LC2 LC3	2.21 1.13 1.19	N/A	N/A	(III)	LC1 LC2 LC3	2.33 1.30 1.30	N/A	N/A

Table 3.18 ULS collapse load factor and deflection at SLS for semi-rigid 4 and 6 bay frames

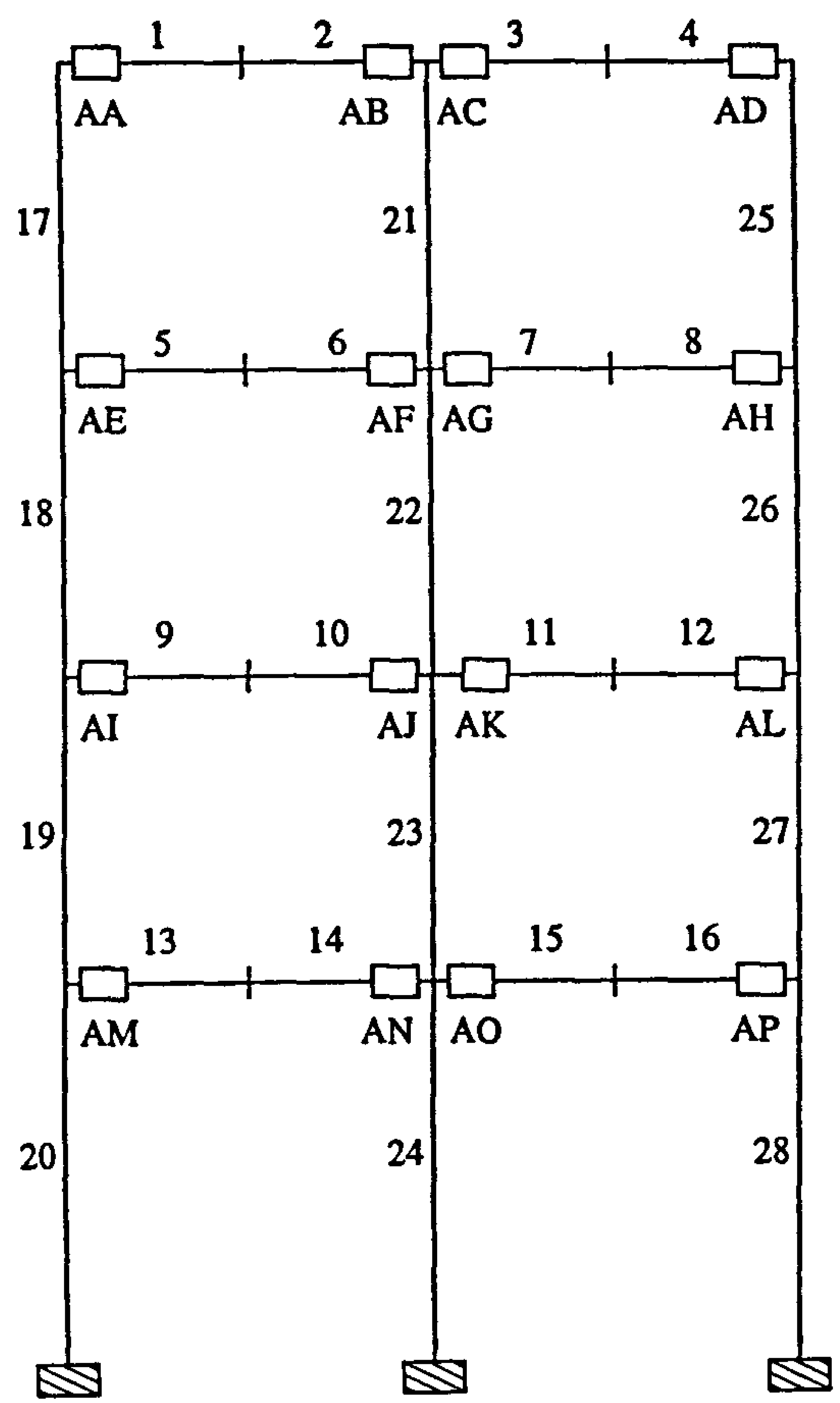
Semi-Rigid Frame (Section Designation II)



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
12	0.53	0.49	0.35	0.44	0.51	0.44
14	0.43	0.40	0.27	0.57	0.45	0.34

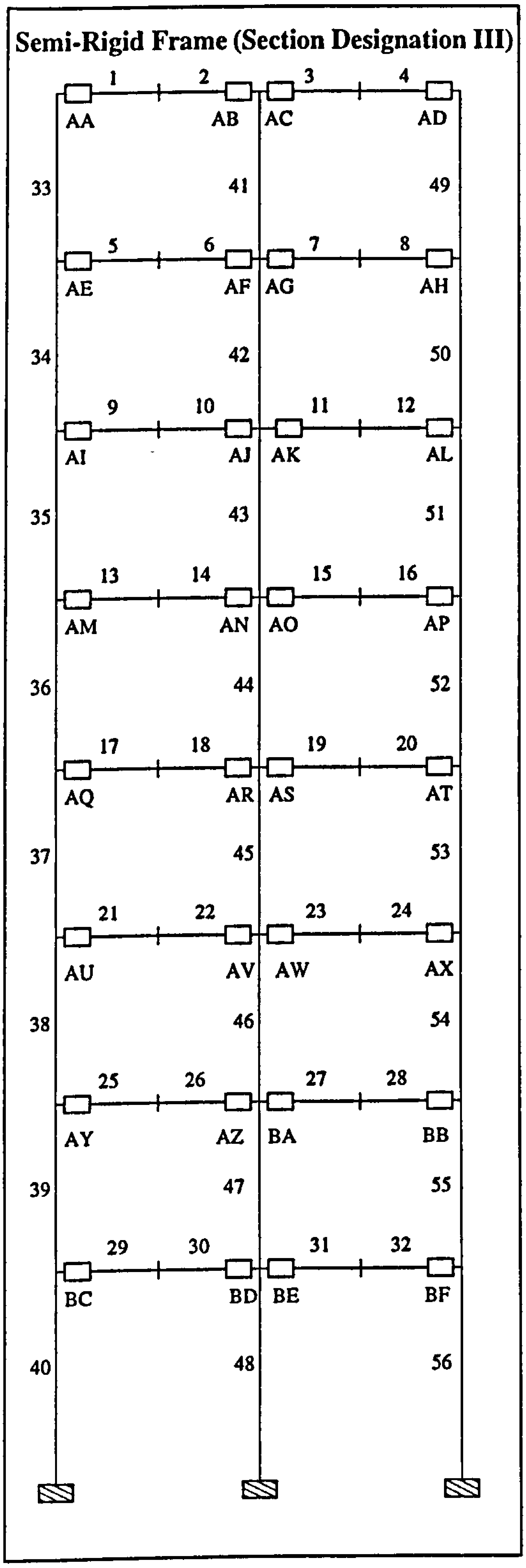
Table 3.19 Frame Identification : Frame 1

Semi-Rigid Frame (Section Designation II)



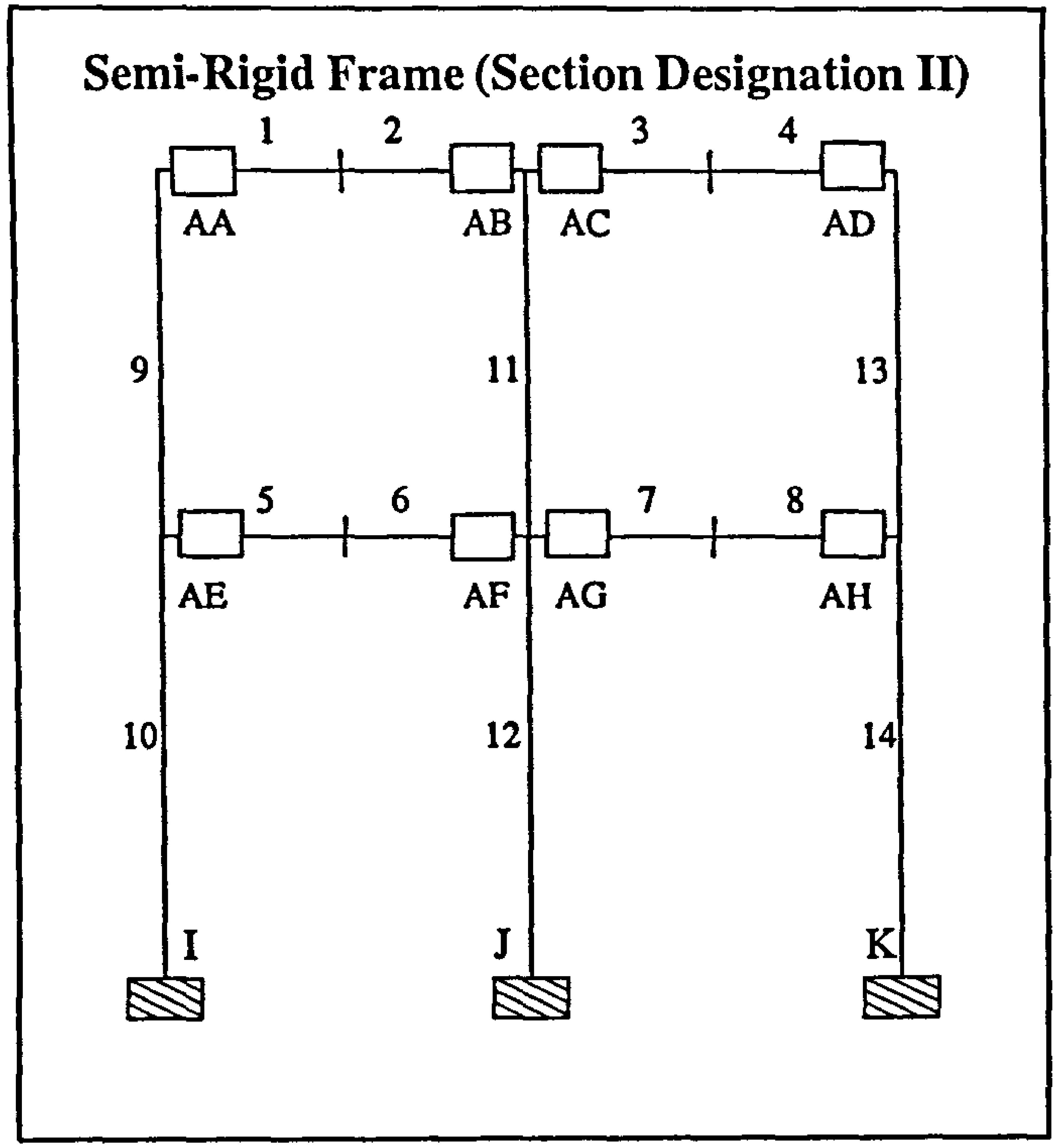
Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
22	0.55	0.67	0.43	0.42	0.56	0.50
24	0.75	0.75	0.56	0.71	0.91	0.80
26	0.51	0.51	0.36	0.58	0.67	0.48
28	0.60	0.62	0.54	0.58	0.75	0.65

Table 3.20 Frame Identification : Frame 2



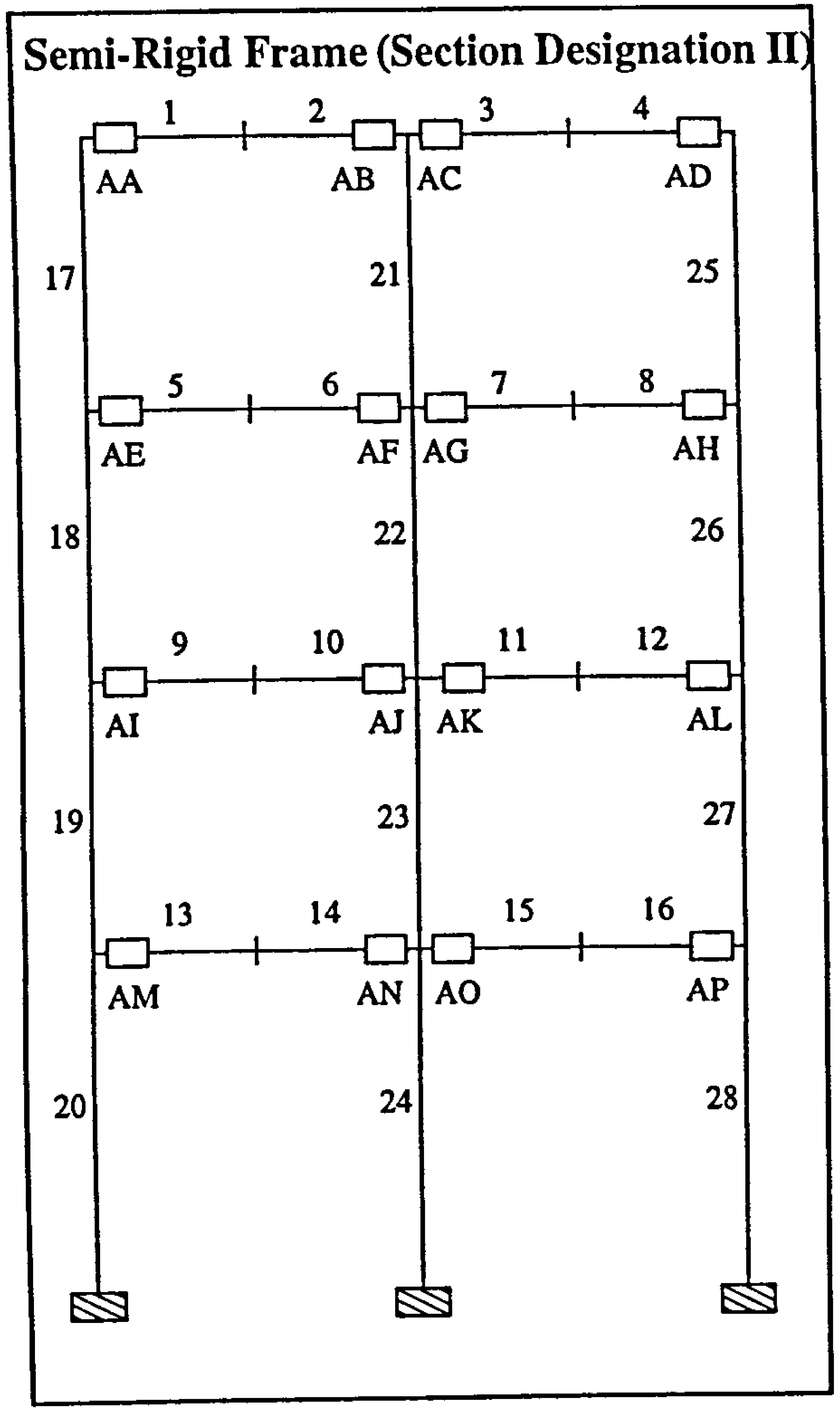
Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
42	0.31	0.39	0.32	0.30	0.56	0.54
45	0.34	0.38	0.31	0.33	0.46	0.42
48	0.63	0.69	0.56	0.56	0.81	0.74
50	0.44	0.50	0.32	0.49	0.70	0.46
53	0.39	0.44	0.36	0.43	0.57	0.49
56	0.56	0.67	0.56	0.53	0.77	0.71

Table 3.21 Frame Identification : Frame 3



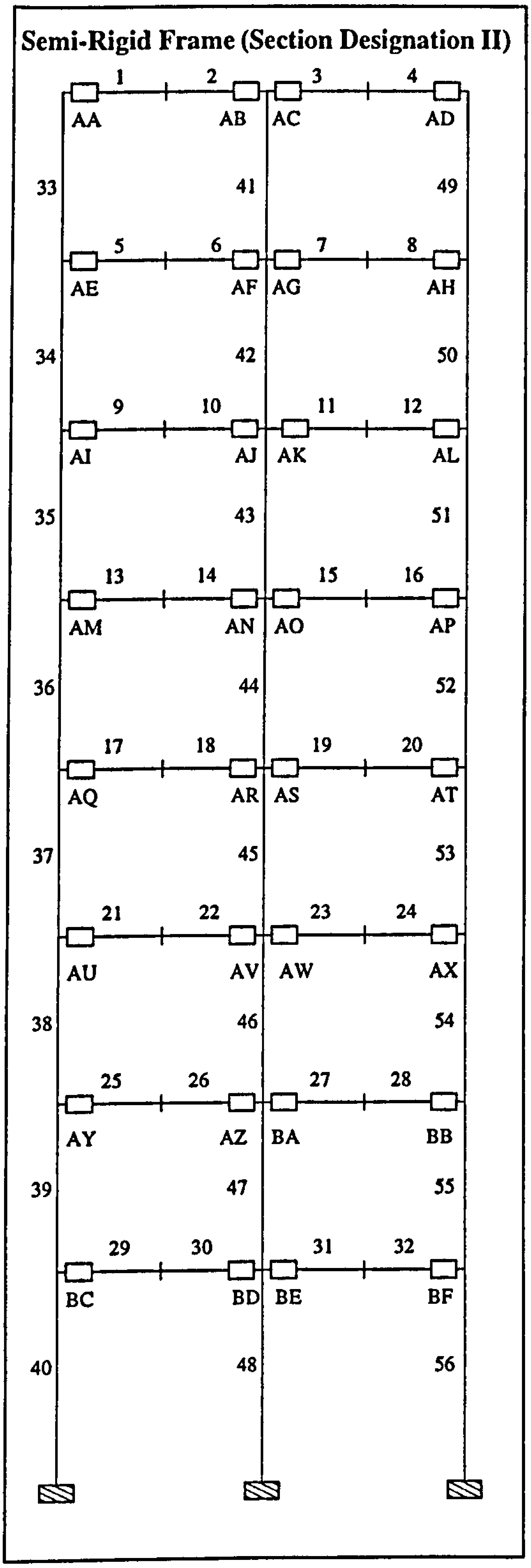
Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
12	0.70	0.78	0.58	0.79	1.07	0.86
14	0.65	0.67	0.47	0.48	0.64	0.52

Table 3.22 Frame Identification : Frame 4



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
22	0.49	0.55	0.39	0.51	0.77	0.61
24	0.83	0.92	0.70	0.90	1.29	1.09
26	0.39	0.45	0.33	0.35	0.53	0.42
28	0.65	0.75	0.58	0.67	0.99	0.85

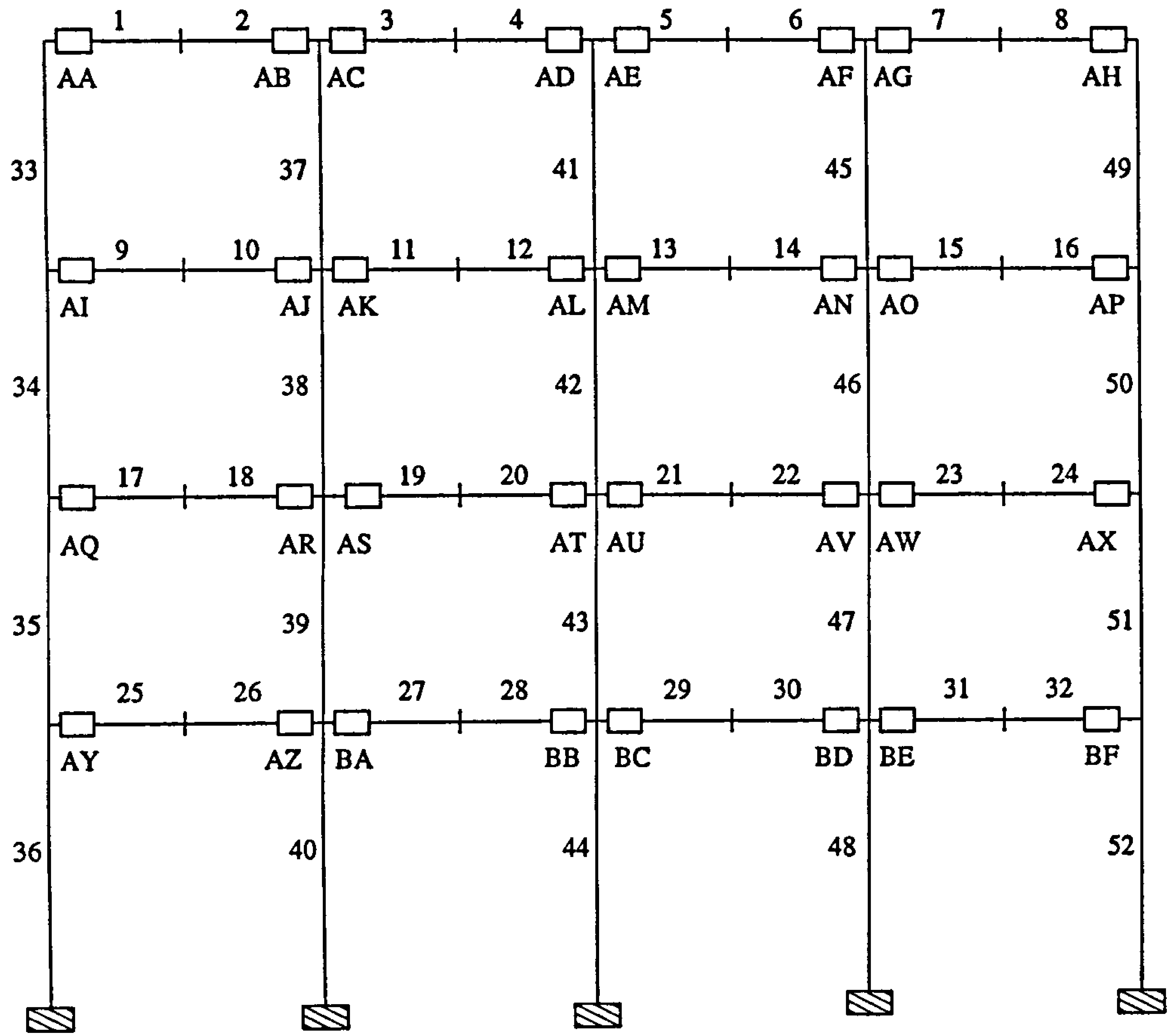
Table 3.23 Frame Identification : Frame 5



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
42	0.31	0.44	0.36	0.31	0.69	0.64
45	0.49	0.58	0.47	0.47	0.73	0.65
48	0.78	0.88	0.71	0.74	1.10	0.99
50	0.29	0.39	0.32	0.23	0.48	0.44
53	0.42	0.53	0.44	0.43	0.74	0.68
56	0.68	0.86	0.72	0.63	1.03	0.94

Table 3.24 Frame Identification : Frame 6

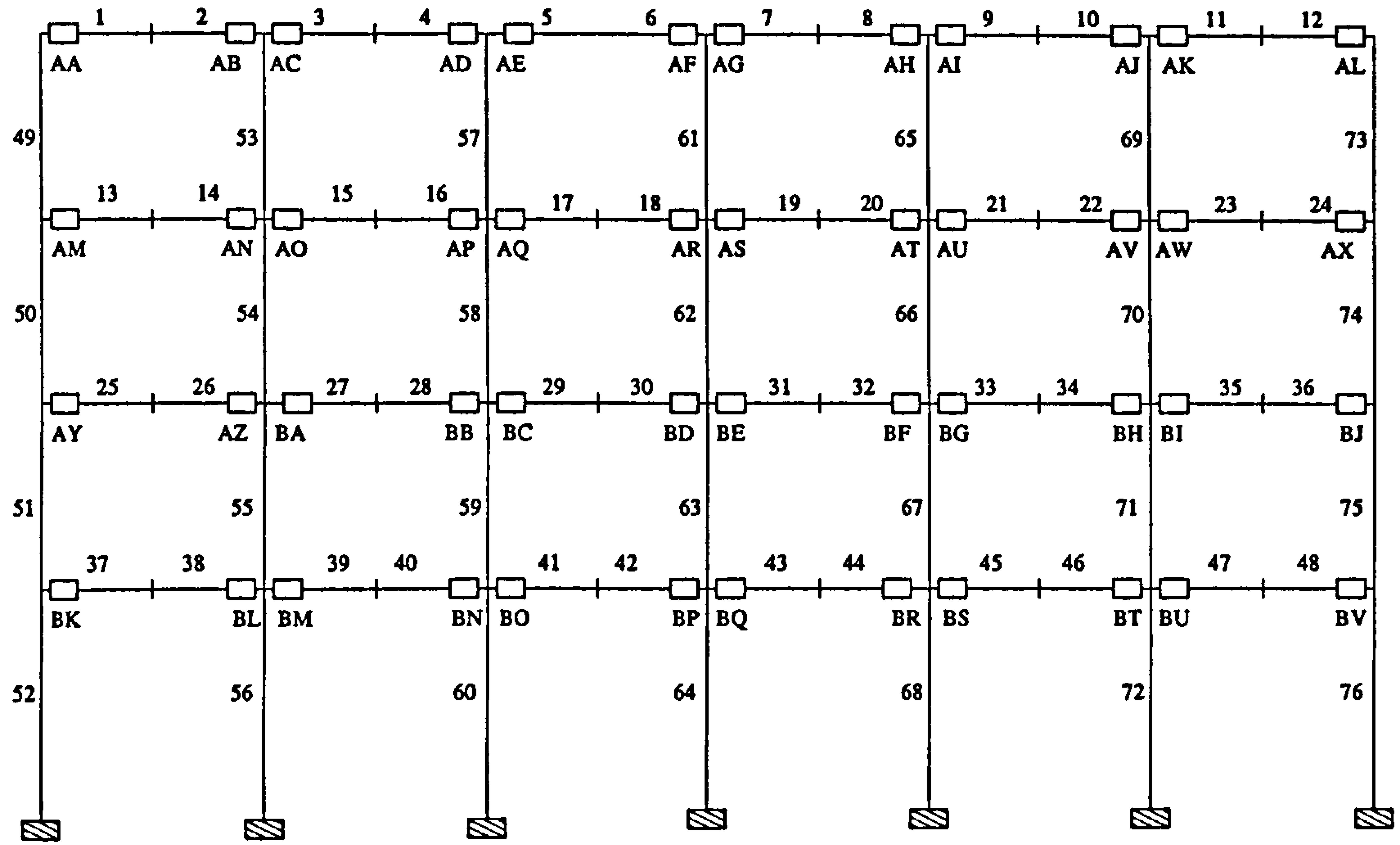
Semi-Rigid Frame (Section Designation III)



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
46	0.57	0.50	0.35	0.46	0.44	0.35
48	0.75	0.63	0.44	0.76	0.67	0.53
50	0.53	0.44	0.33	0.63	0.53	0.38
52	0.63	0.52	0.40	0.62	0.54	0.46

Table 3.25 Frame Identification : Frame 7

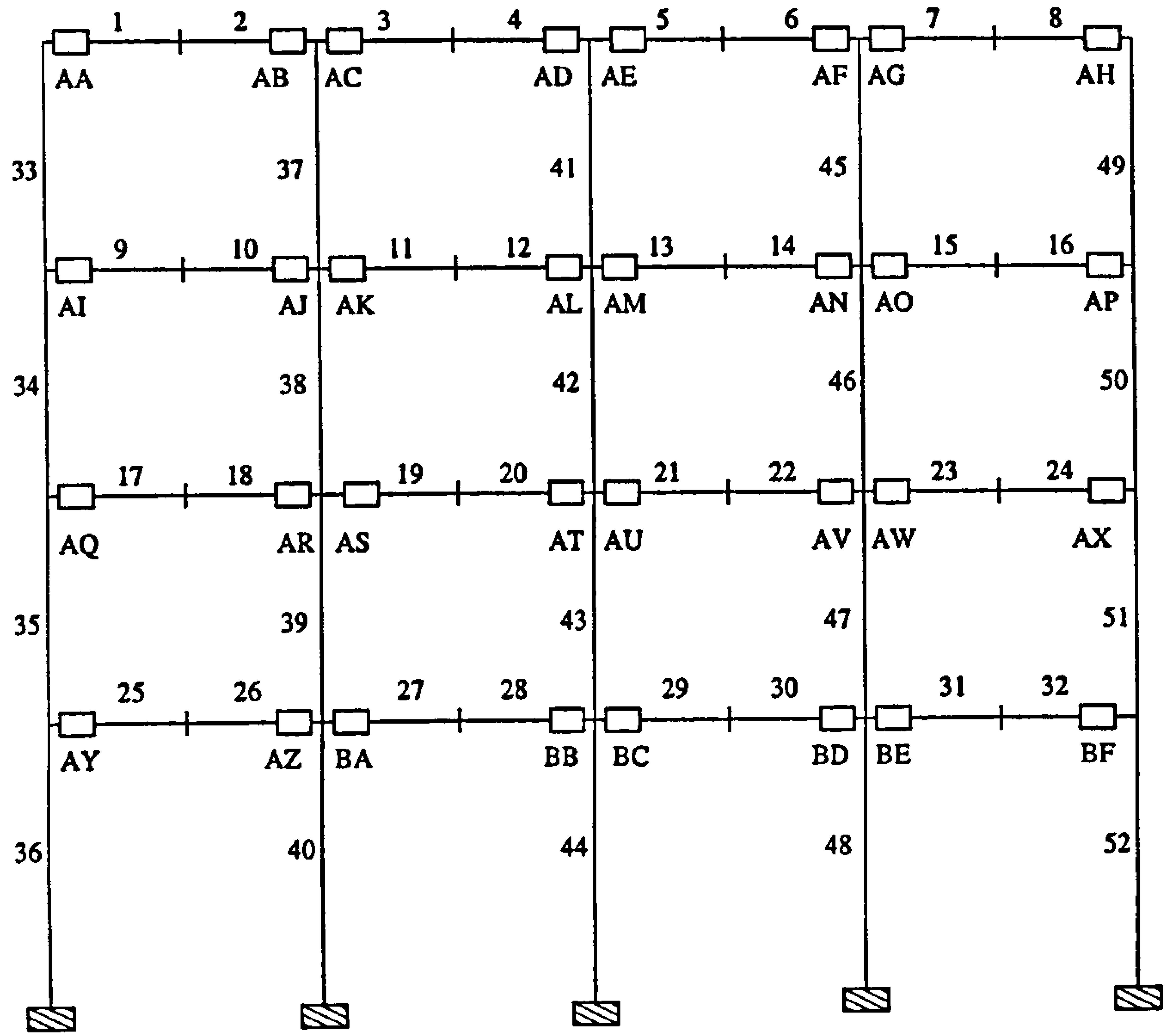
Semi-Rigid Frame (Section Designation II)



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
70	0.58	0.46	0.31	0.47	0.37	0.28
72	0.76	0.59	0.40	0.77	0.57	0.44
74	0.55	0.42	0.30	0.66	0.50	0.33
76	0.61	0.44	0.39	0.65	0.42	0.40

Table 3.26 Frame Identification : Frame 8

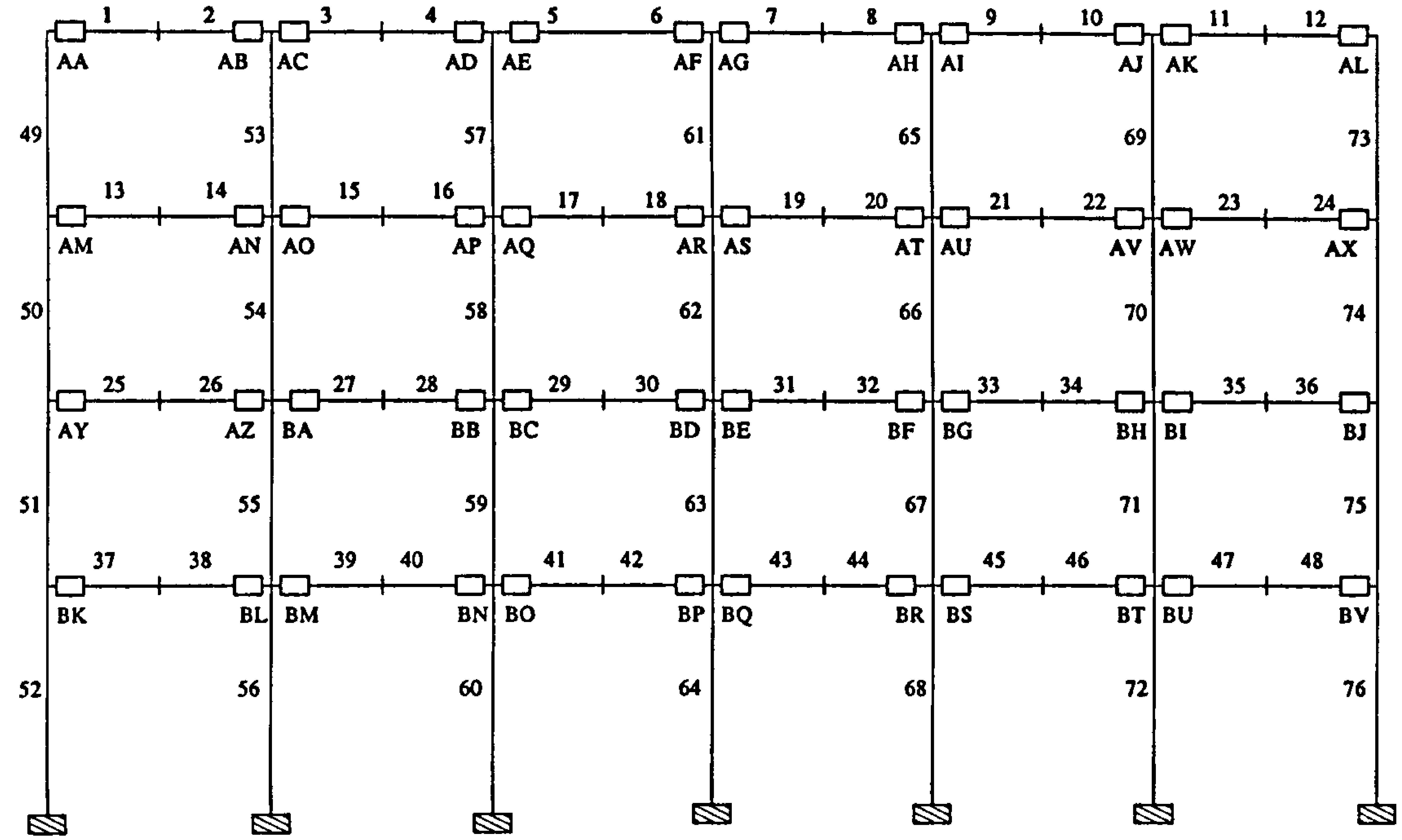
Semi-Rigid Frame (Section Designation IV)



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
46	0.60	0.54	0.38	0.51	0.53	0.41
48	0.75	0.64	0.45	0.74	0.68	0.55
50	0.54	0.48	0.33	0.37	0.39	0.31
52	0.58	0.50	0.36	0.53	0.49	0.41

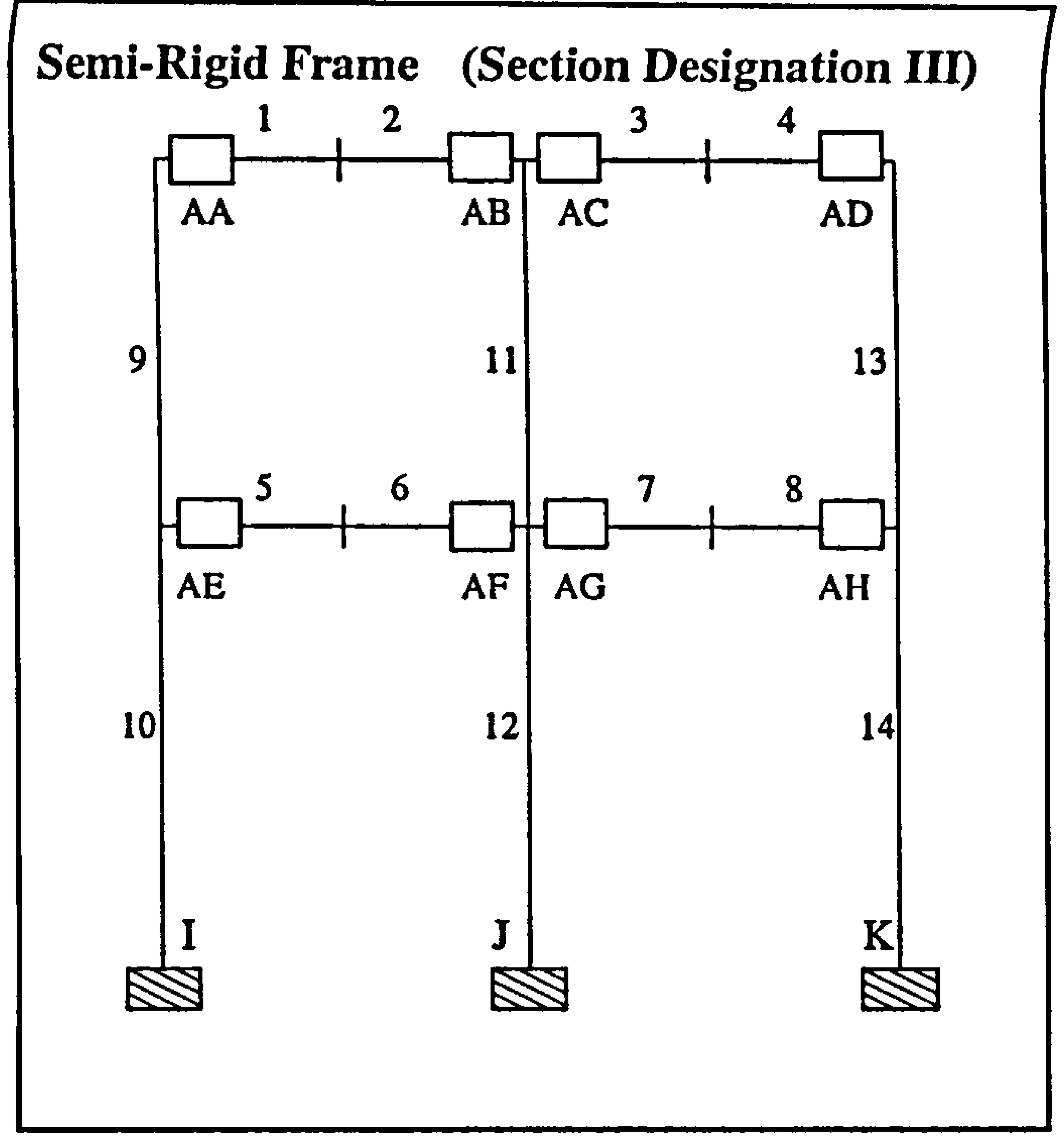
Table 3.27 Frame Identification : Frame 9

Semi-Rigid Frame (Section Designation IV)



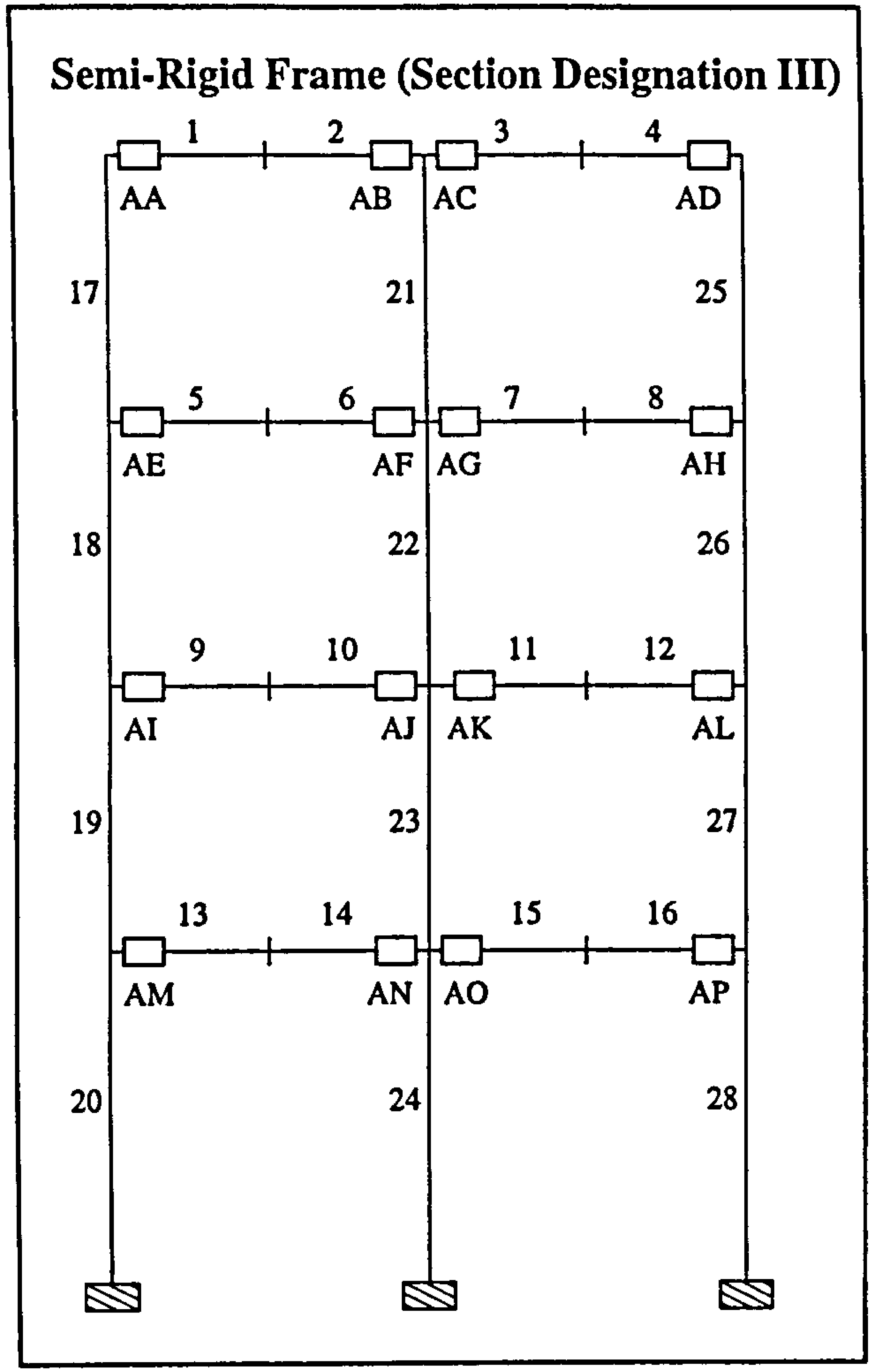
Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
70	0.57	0.47	0.32	0.47	0.40	0.30
72	0.76	0.60	0.41	0.76	0.58	0.45
74	0.56	0.47	0.32	0.42	0.36	0.29
76	0.68	0.53	0.38	0.75	0.56	0.45

Table 3.28 Frame Identification : Frame 10



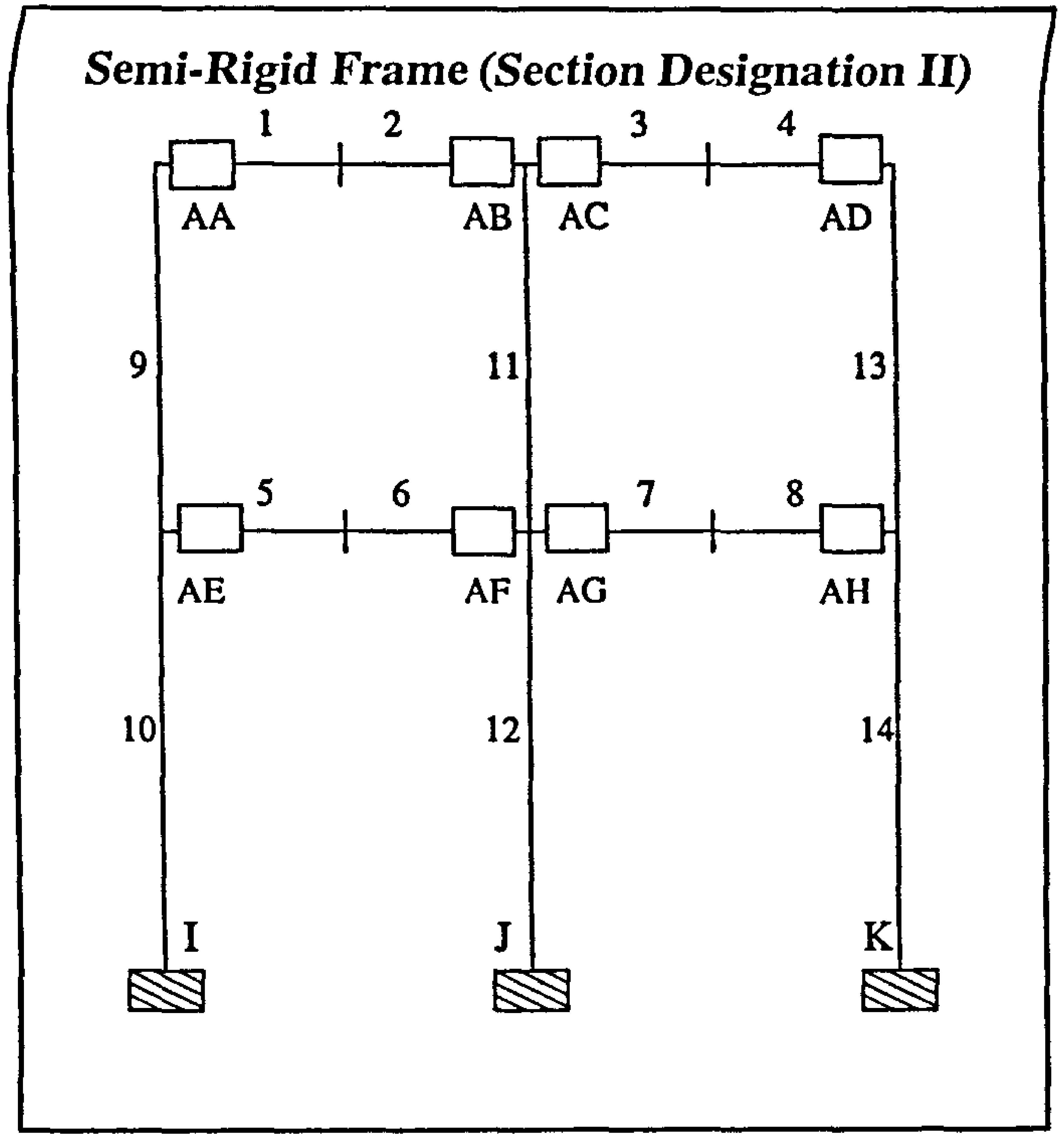
Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
12	0.21	0.44	0.42	0.19	0.67	0.69
14	0.20	0.38	0.36	0.20	0.56	0.57

Table 3.29 Frame Identification : Frame 11



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
22	0.09	0.20	0.20	0.09	0.31	0.33
24	0.17	0.40	0.40	0.14	0.57	0.62
26	0.13	0.22	0.21	0.16	0.34	0.34
28	0.13	0.34	0.36	0.12	0.52	0.57

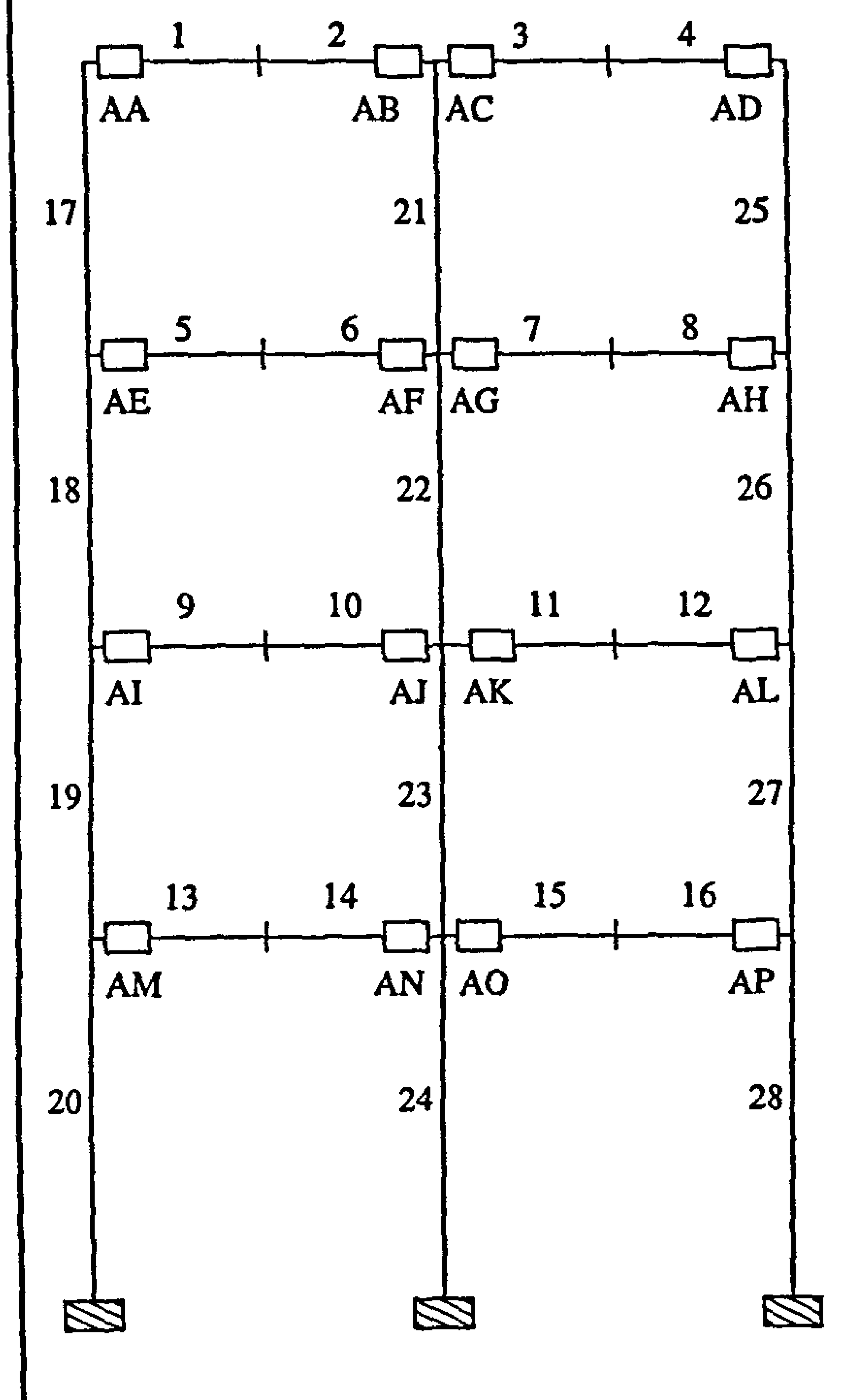
Table 3.30 Frame Identification : Frame 12



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
12	0.24	0.57	0.57	0.22	1.04	1.13
14	0.19	0.46	0.46	0.16	0.82	0.89

Table 3.31 Frame Identification : Frame 14

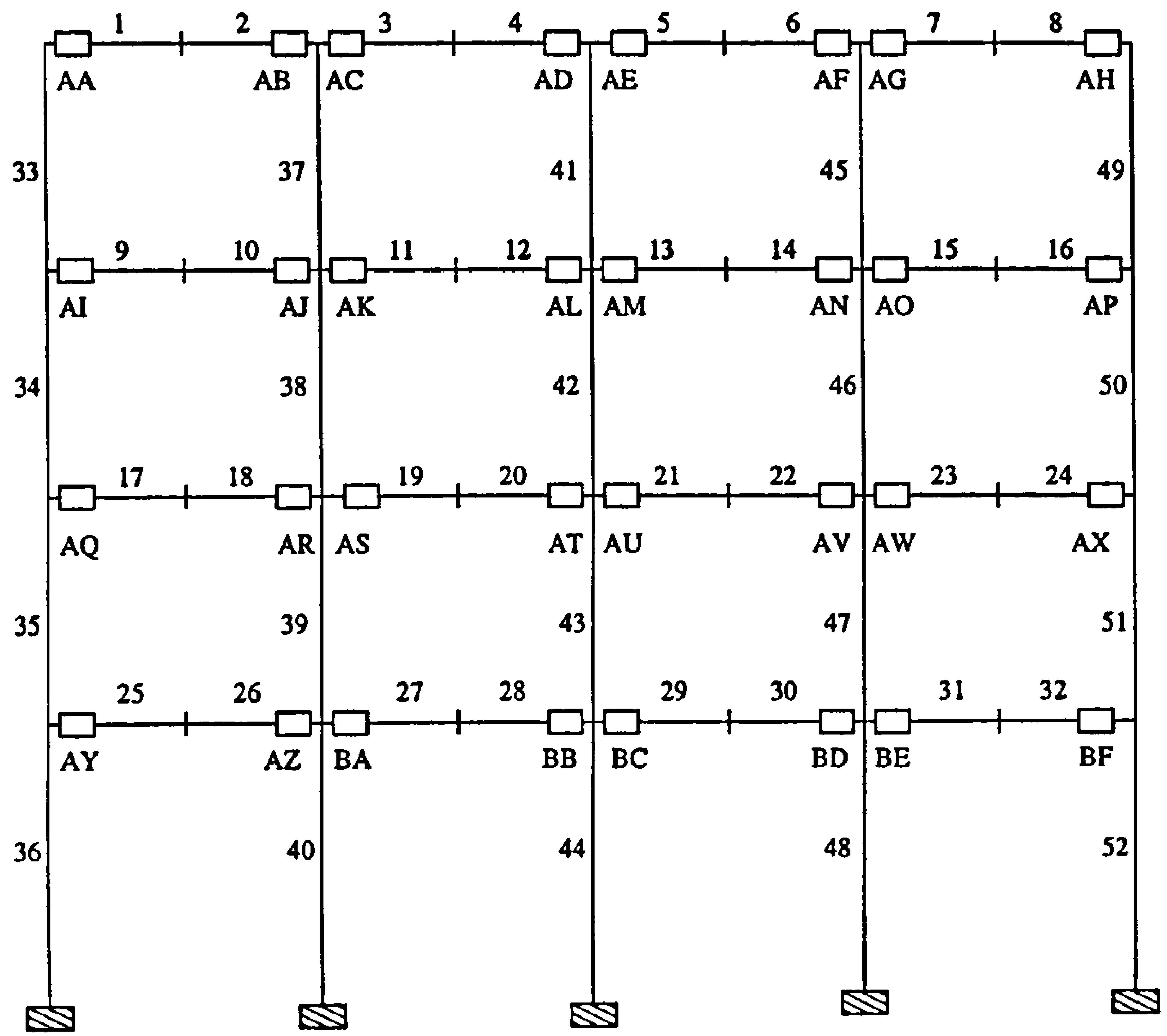
Semi-Rigid Frame (Section Designation II)



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
22	0.13	0.30	0.28	0.12	0.47	0.47
24	0.20	0.51	0.53	0.18	0.82	0.90
26	0.11	0.28	0.30	0.10	0.50	0.59
28	0.20	0.54	0.54	0.17	0.81	0.87

Table 3.32 Frame Identification : Frame 15

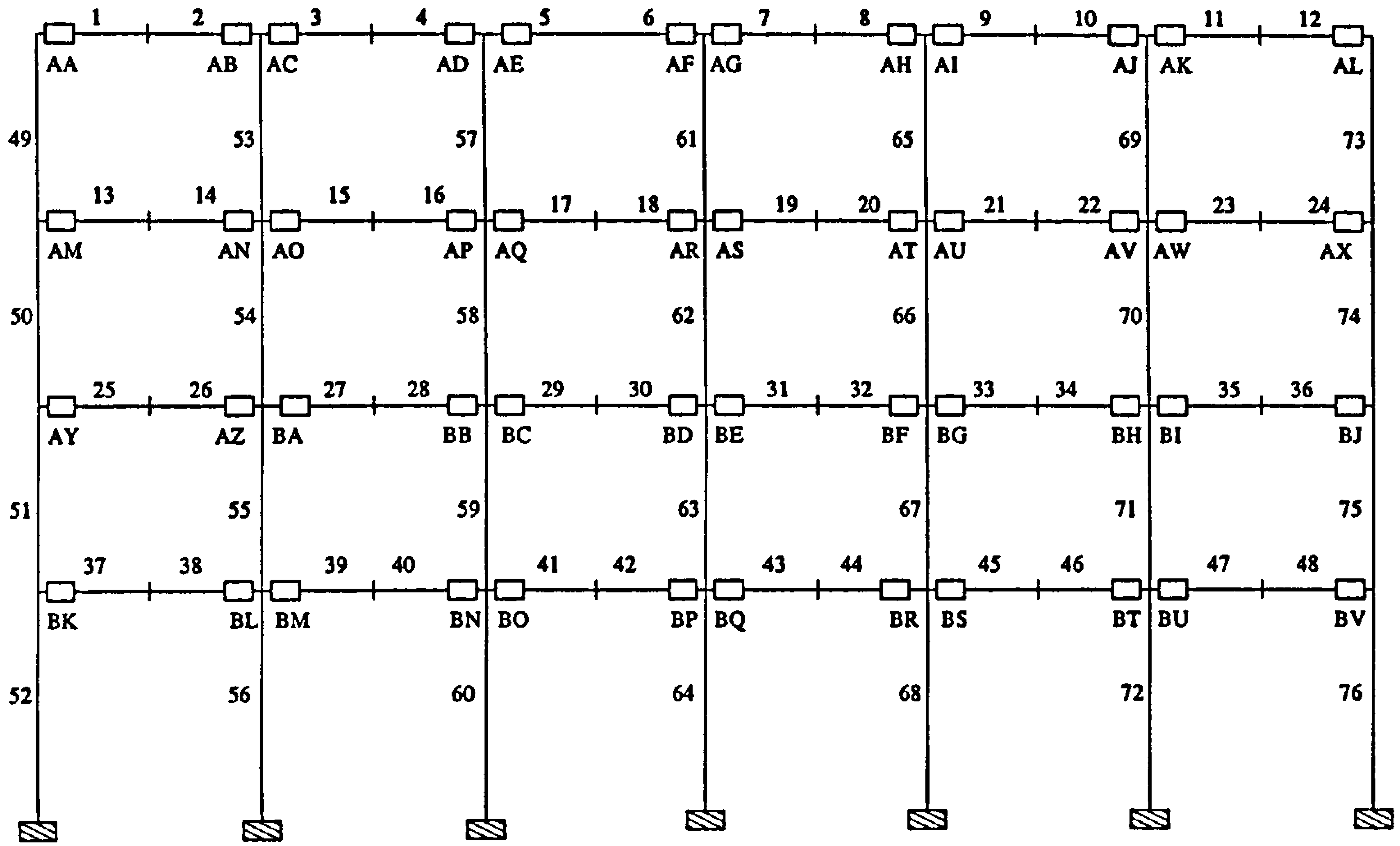
Semi-Rigid Frame (Section Designation III)



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
46	0.19	0.29	0.26	0.16	0.42	0.40
48	0.26	0.42	0.40	0.24	0.60	0.62
50	0.20	0.18	0.21	0.24	0.36	0.32
52	0.26	0.49	0.41	0.26	0.68	0.71

Table 3.33 Frame Identification : Frame 17

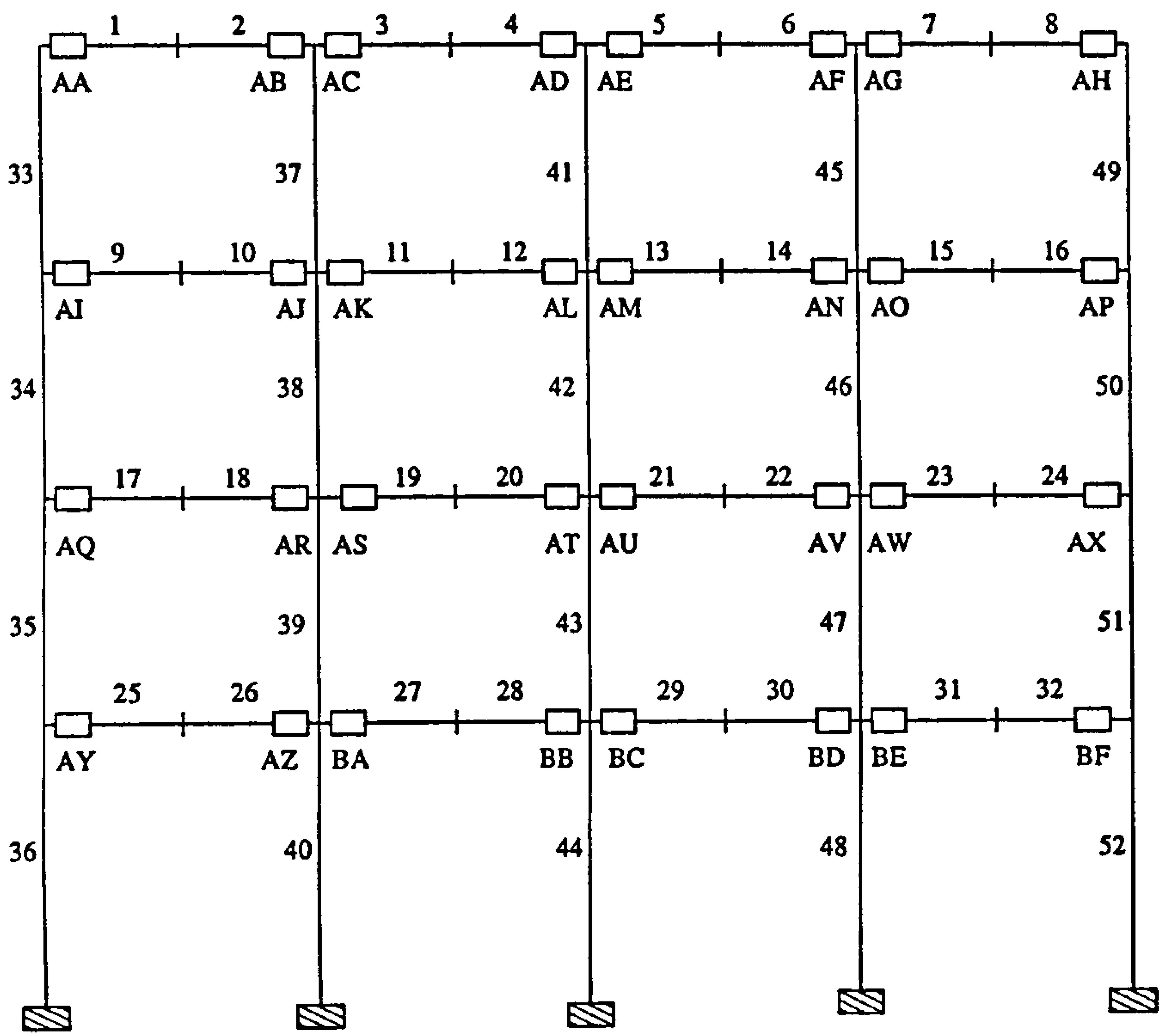
Semi-Rigid Frame (Section Designation III)



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
70	0.21	0.28	0.24	0.20	0.41	0.40
72	0.40	0.53	0.48	0.35	0.69	0.69
74	0.28	0.34	0.28	0.43	0.60	0.50
76	0.34	0.52	0.81	0.34	0.79	0.81

Table 3.34 Frame Identification : Frame 18

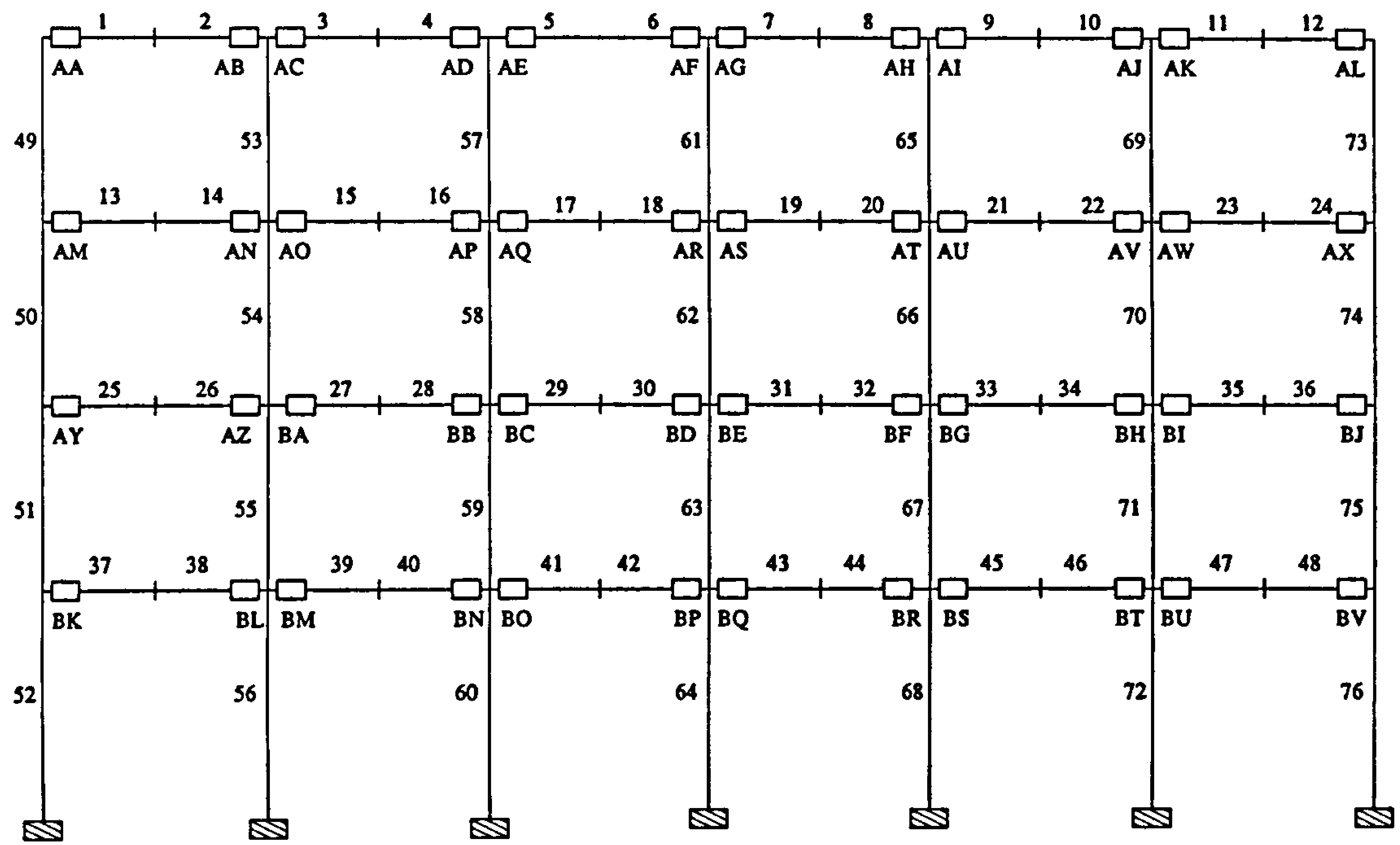
Semi-Rigid Frame (Section Designation II)



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
46	0.22	0.35	0.31	0.21	0.56	0.58
48	0.40	0.69	0.66	0.36	0.99	1.03
50	0.16	0.27	0.27	0.14	0.44	0.46
52	0.31	0.59	0.58	0.25	0.81	0.85

Table 3.35 Frame Identification : Frame 19

Semi-Rigid Frame (Section Designation II)



Member	Stability Factors					
	Overall buckling check			Local capacity check		
	LC1	LC2	LC3	LC1	LC2	LC3
70	0.22	0.37	0.36	0.23	0.49	0.47
72	0.49	0.73	0.66	0.49	1.16	1.17
74	0.26	0.40	0.35	0.22	0.52	0.50
76	0.45	0.92	0.88	0.49	1.56	1.61

Table 3.36 Frame Identification : Frame 20

	Minimum wind	Maximum wind
Number of storeys	2 to 8	2 to 8
Number of bays	2 to 6	2 to 6
Bay width	6m	6m
Storey height (bottom storey)	6m	6m
Storey height (elsewhere)	5m	5m
Dead load on floors	3.50 kN/m^2	5.00 kN/m^2
Imposed load on floors	4.00 kN/m^2	7.50 kN/m^2
Dead load on roof	3.75 kN/m^2	3.75 kN/m^2
Imposed load on roof	1.50 kN/m^2	1.50 kN/m^2
Basic wind speed	37 m/s	52 m/s
Basic steel grade	S275	S275

Fig 3.1 Range of "wind-moment" study on minor axis

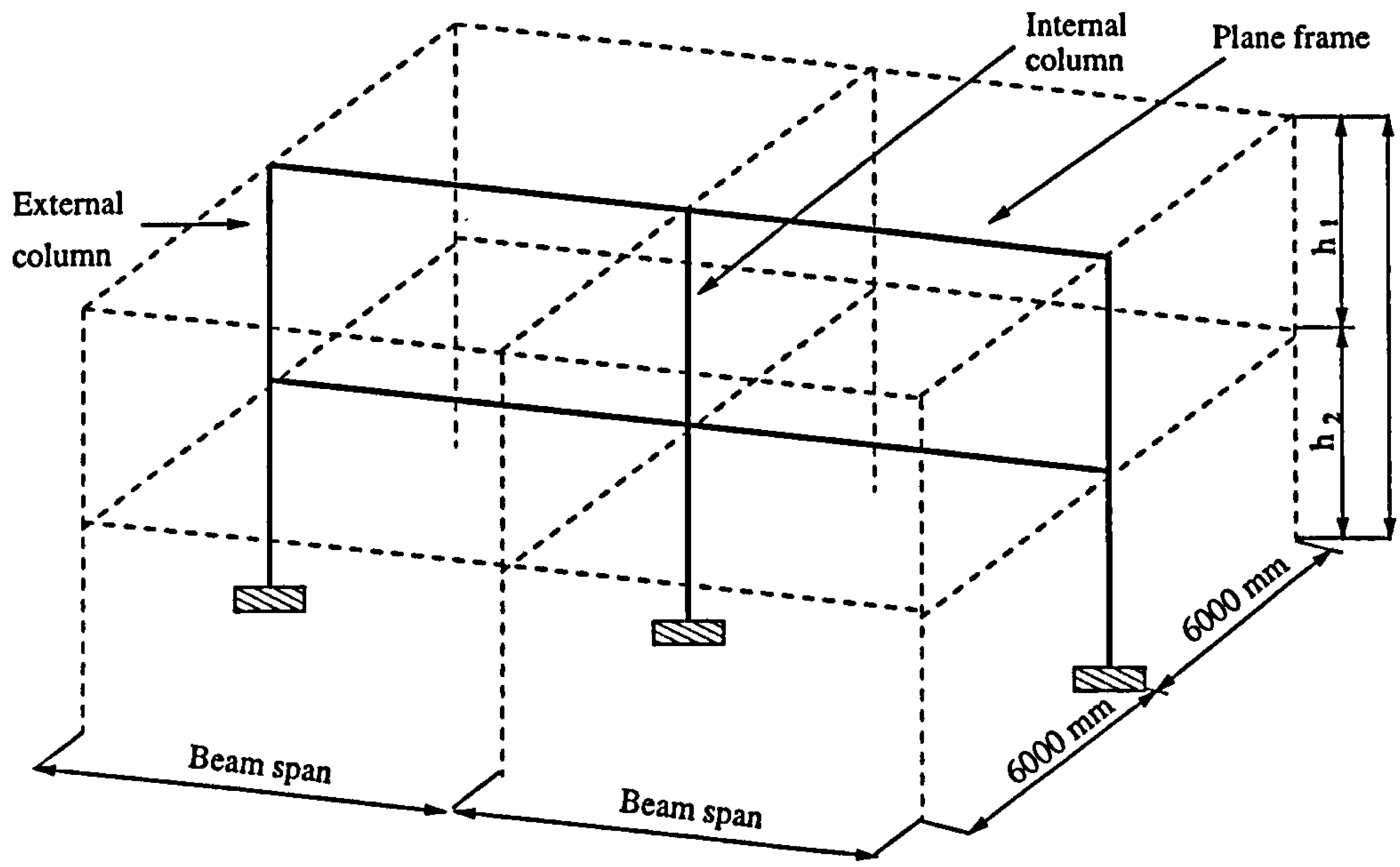


Figure 3.2(a) Typical layout for two bay frames (three dimensions)

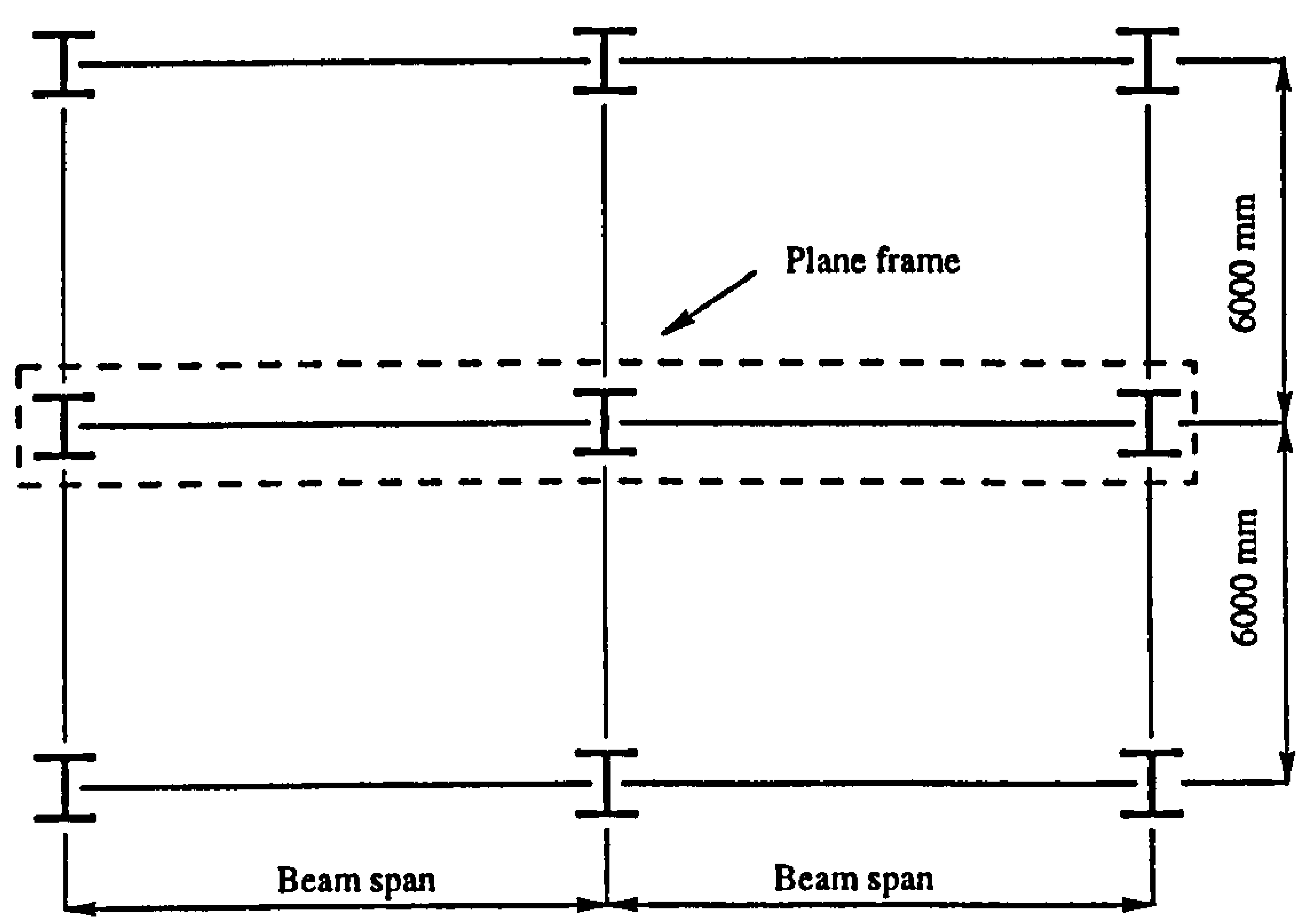
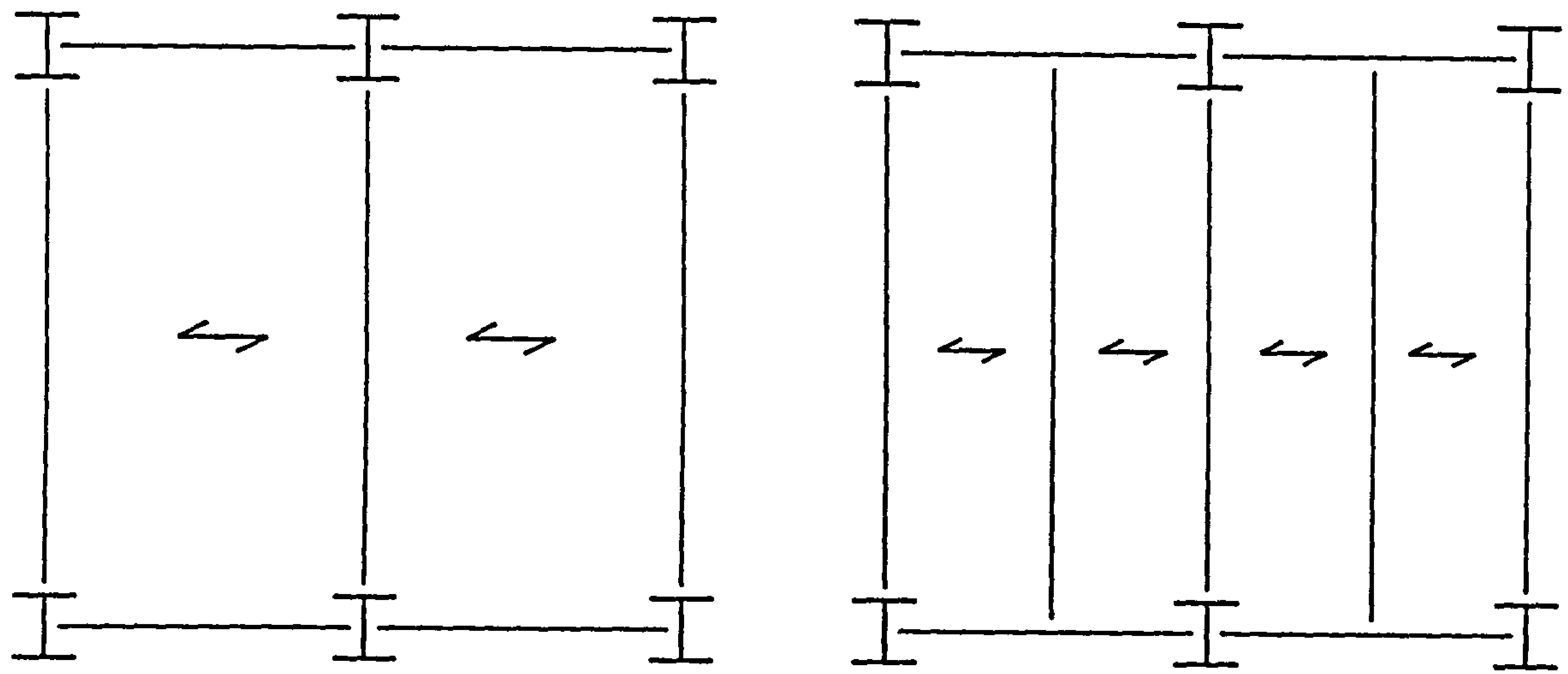


Figure 3.2(b) Typical layout for two bay frames (top view)



a) Precast units

b) Composite floor

Fig. 3.3 Floor grids

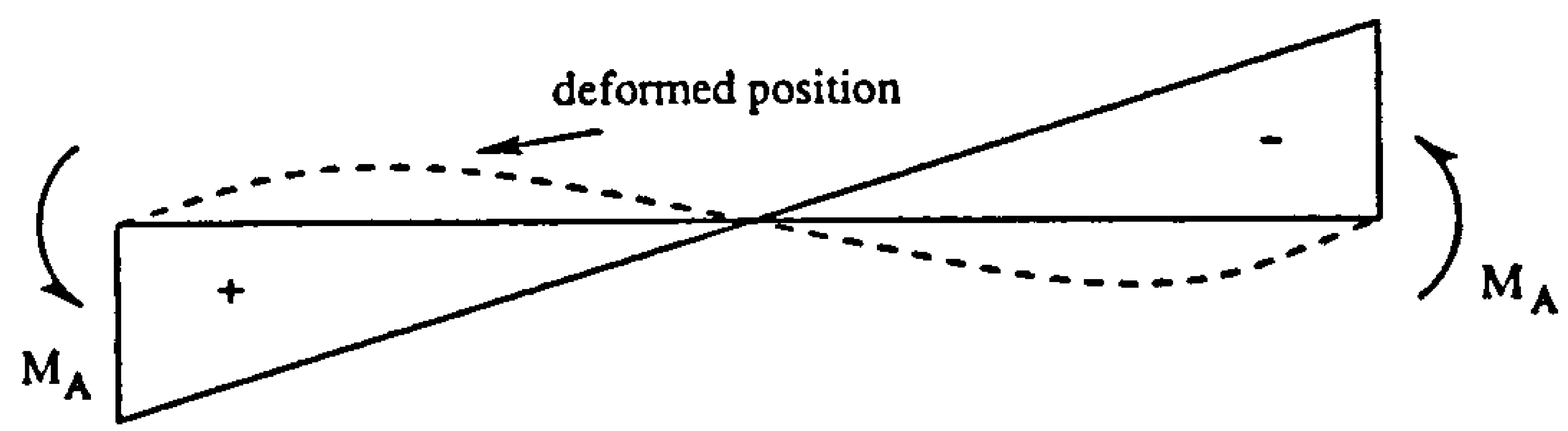


Fig. 3.4 : Bending moment diagram contributes to double curvature

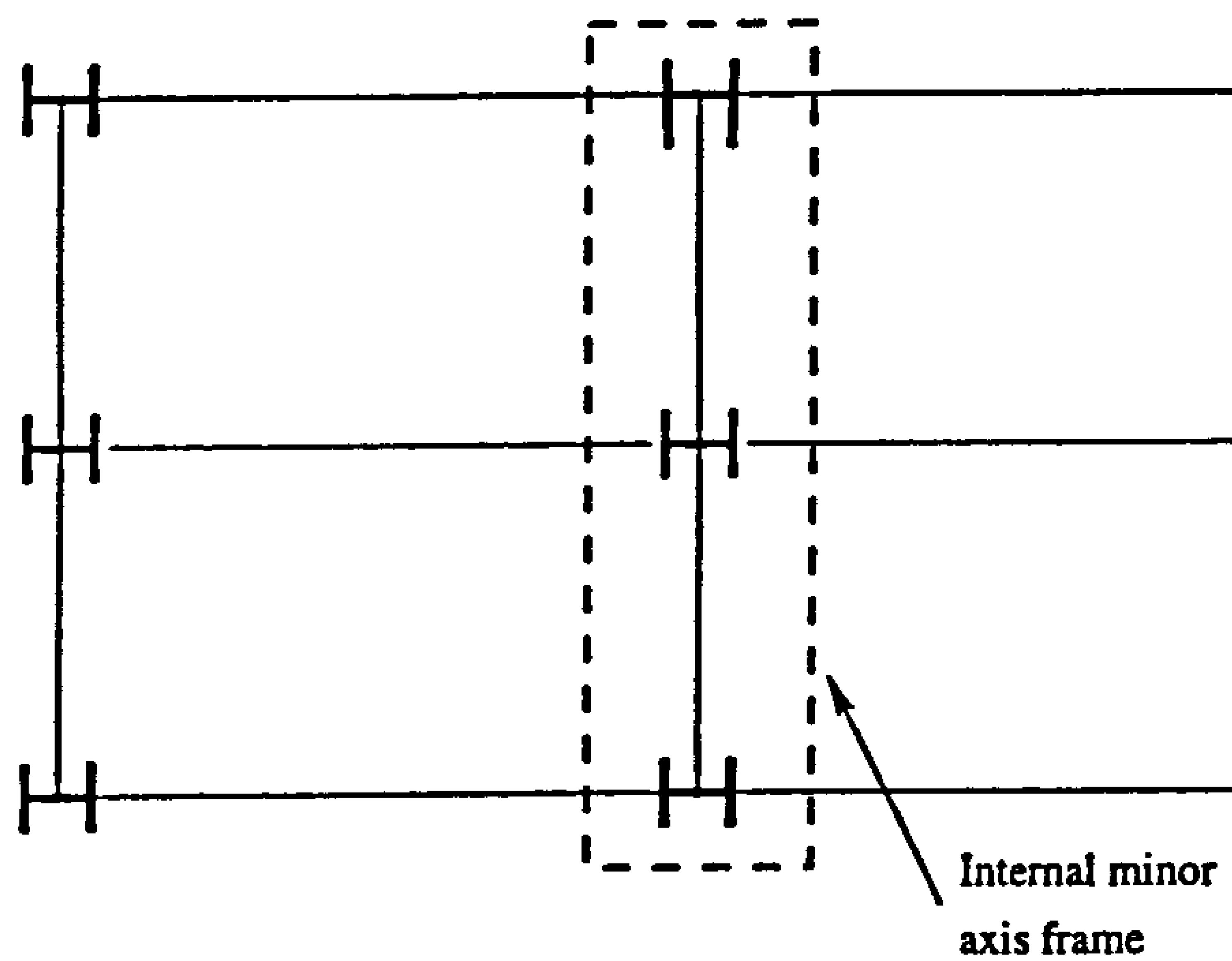
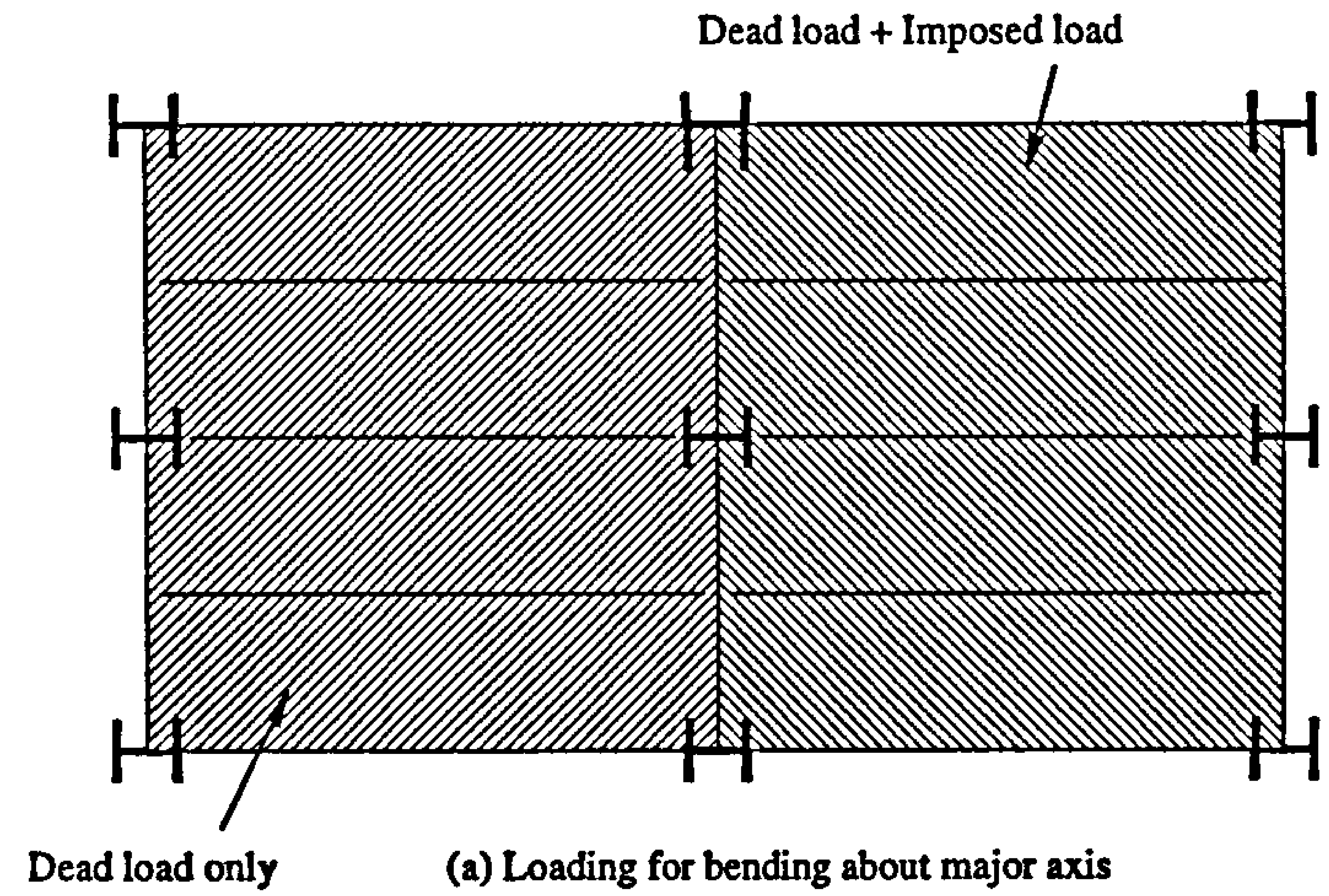
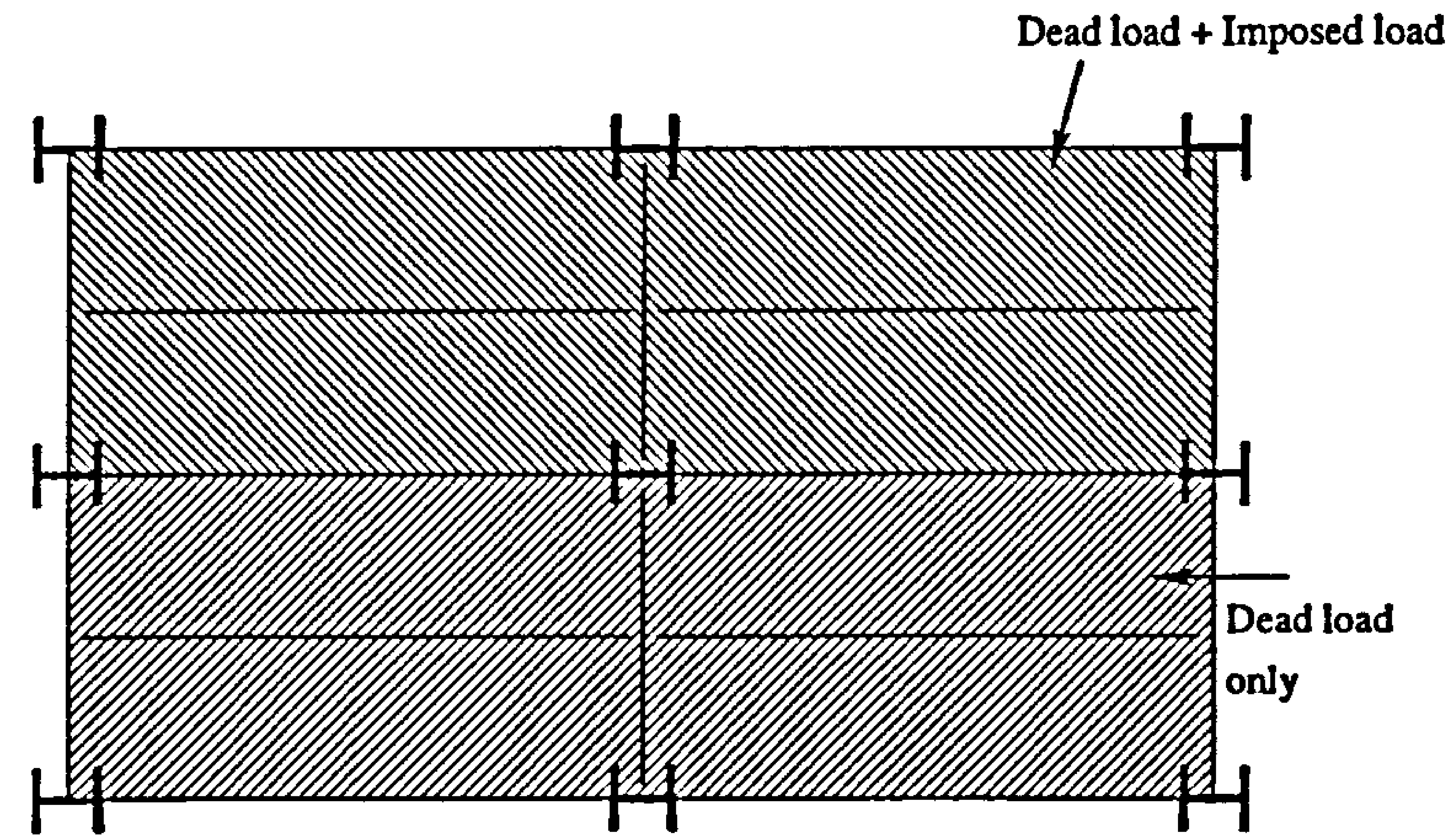


Fig 3.5 Framing plan



(a) Loading for bending about major axis



(b) Loading for bending about minor axis

Fig 3.6 Arrangements of pattern loading

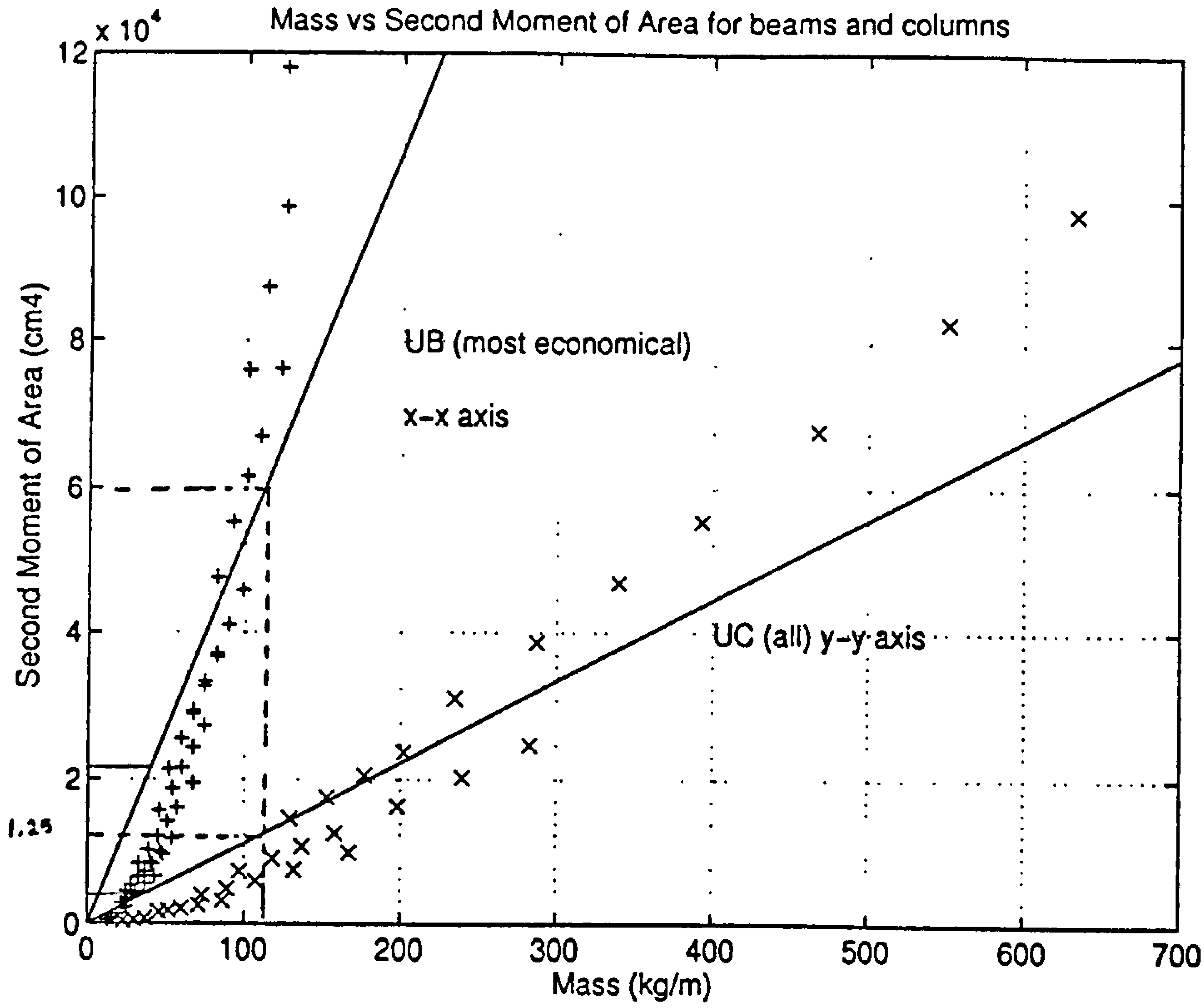


Fig. 3.7 Comparison of I_{x-x} Universal Beam and I_{y-y} Universal Column to calculate optimazation factor k_3 equal to 4.8

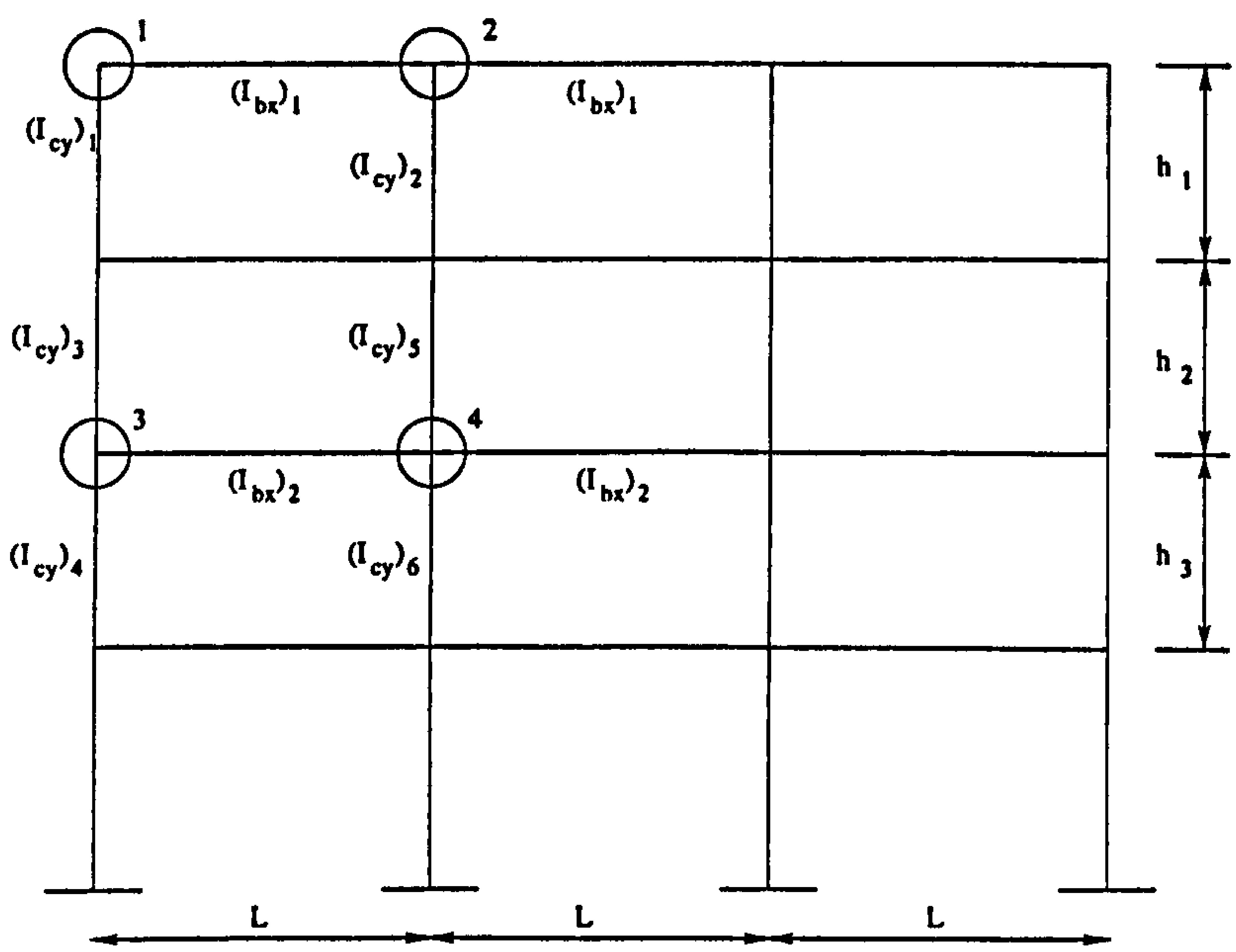


Fig 3.8 Notation for minimum beam stiffness check

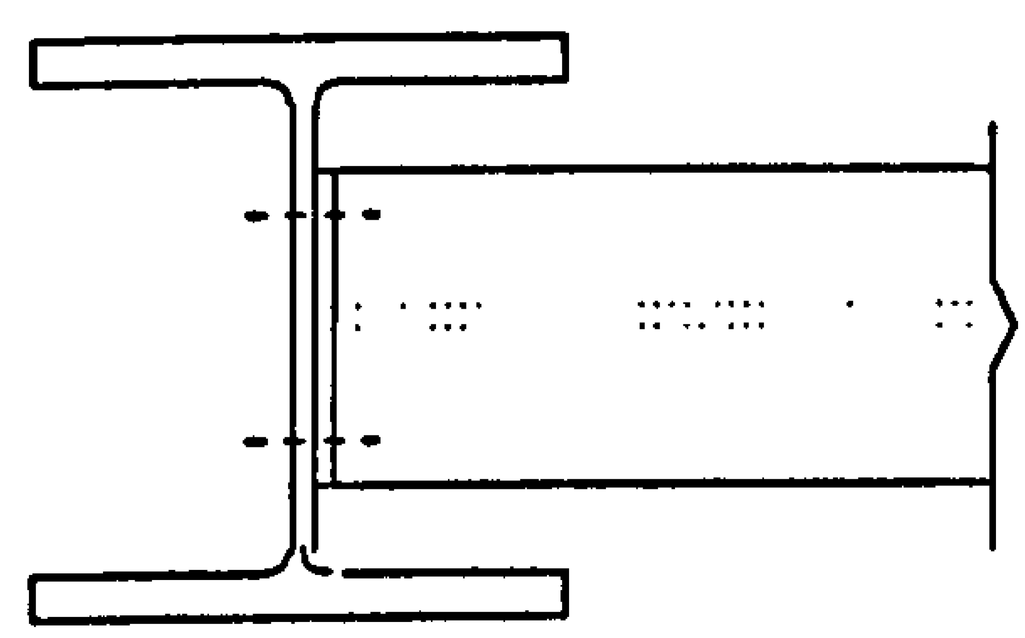


Fig 3.9 Minor axis web connection

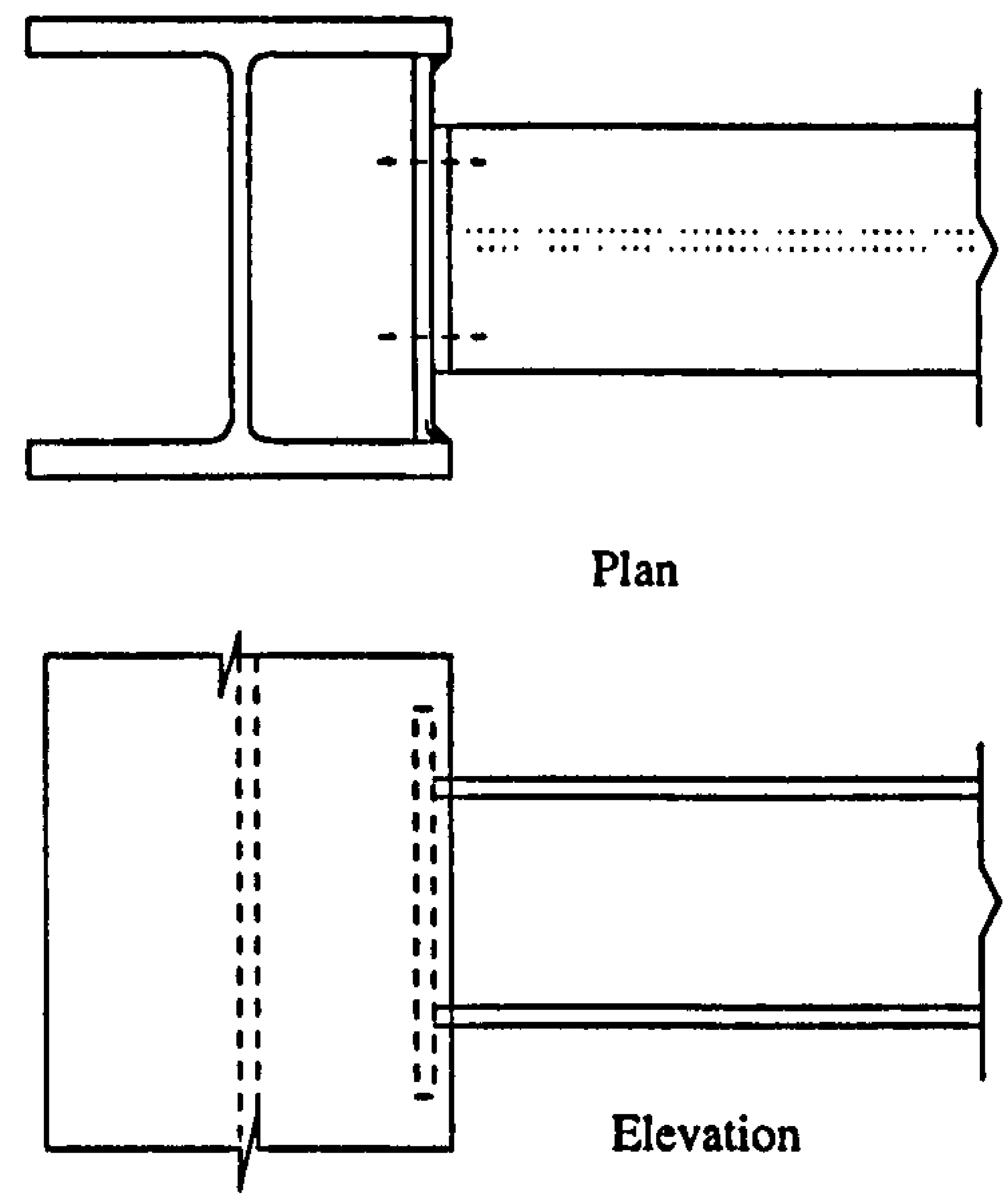


Fig 3.10 Minor axis connections for "wind-moment" frames

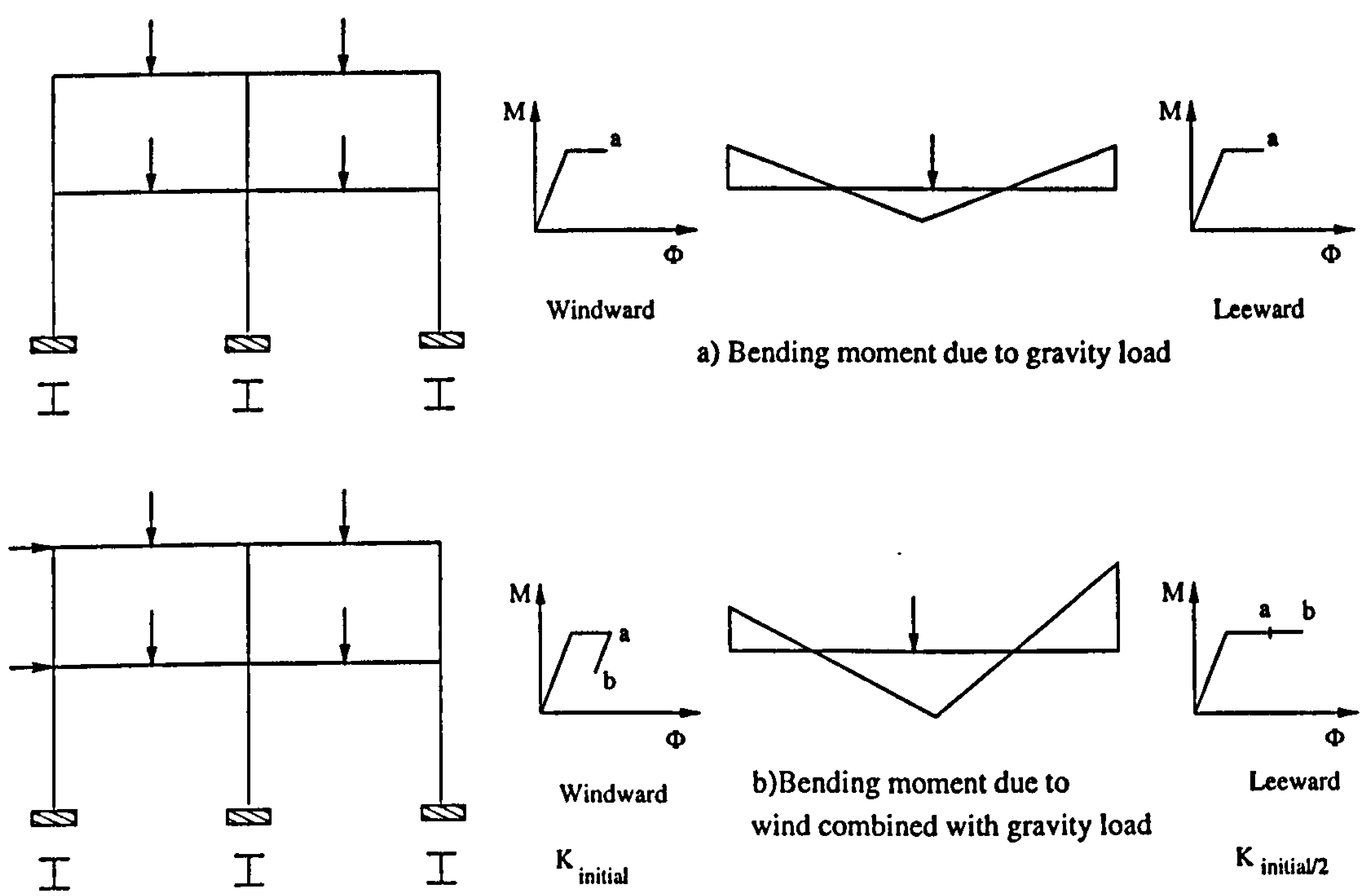


Fig. 3.11 M- Φ curve for connection used for floor grid as shown in Fig 3.3 (b)

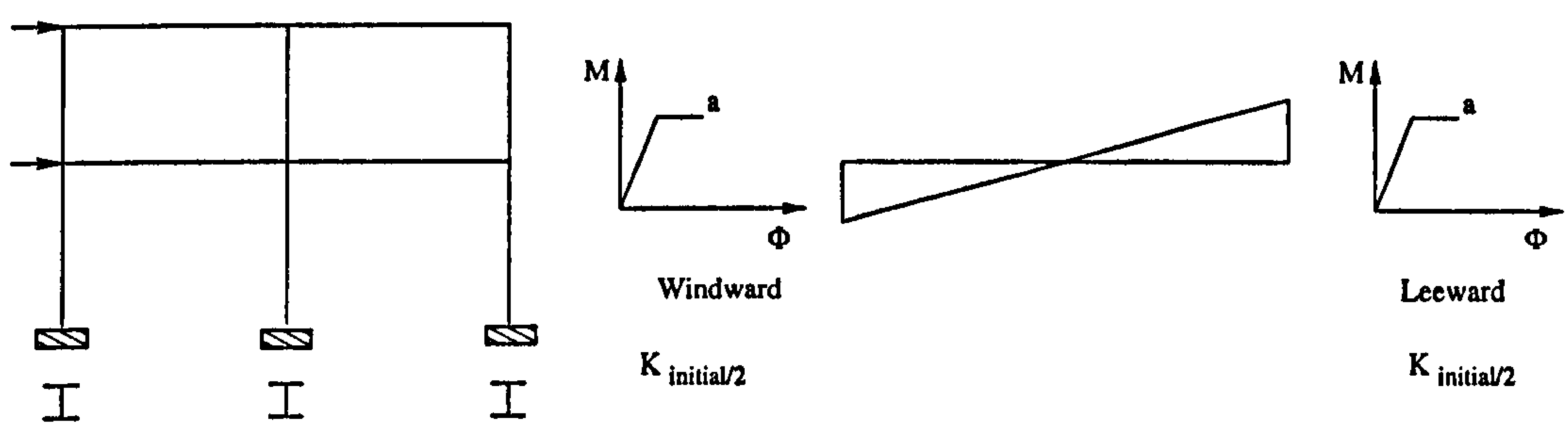
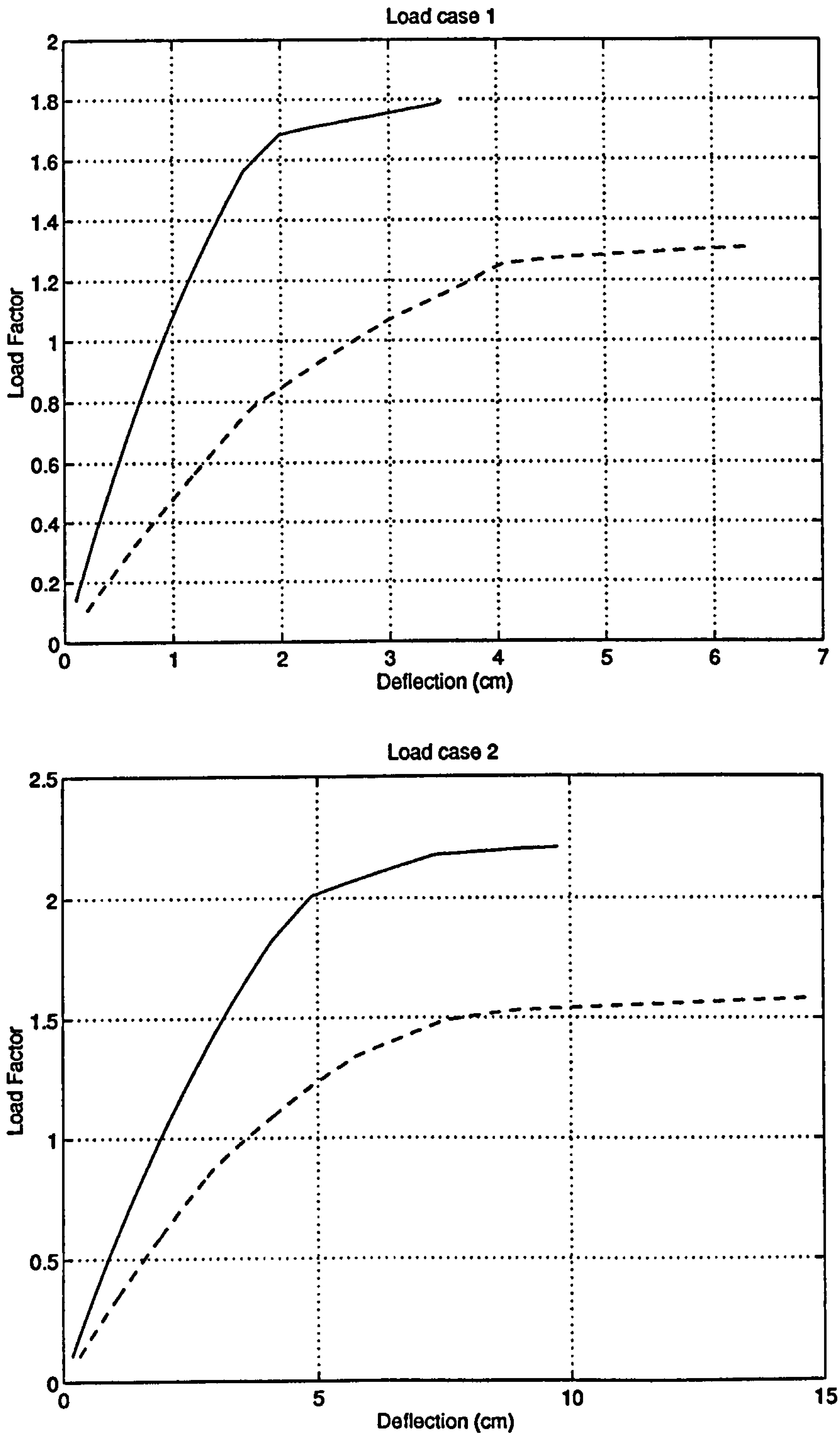


Fig. 3.12 M- Φ curve for connection used for floor grid as shown in Fig 3.3 (a)

Floor grid	Service Limit State		Ultimate Limit State	
	windward	leeward	windward	leeward
Fig 3.2 (a) (wind load only)	$K_{ini.}/2$	$K_{ini.}/2$	$K_{ini.}/2$	$K_{ini.}/2$
Fig 3.2 (b) (wind load combined with gravity load)	$K_{ini.}$	$K_{ini.}/2$	$K_{ini.}$	$K_{ini.}/2$

Fig. 3.13: Summary of stiffness used

Frame 1
Section Desigantion II



Key
Rigid Frame ———
Semi-Rigid Frame - - - - -

Fig. 3.14 (a and b) Load-deflection curves.

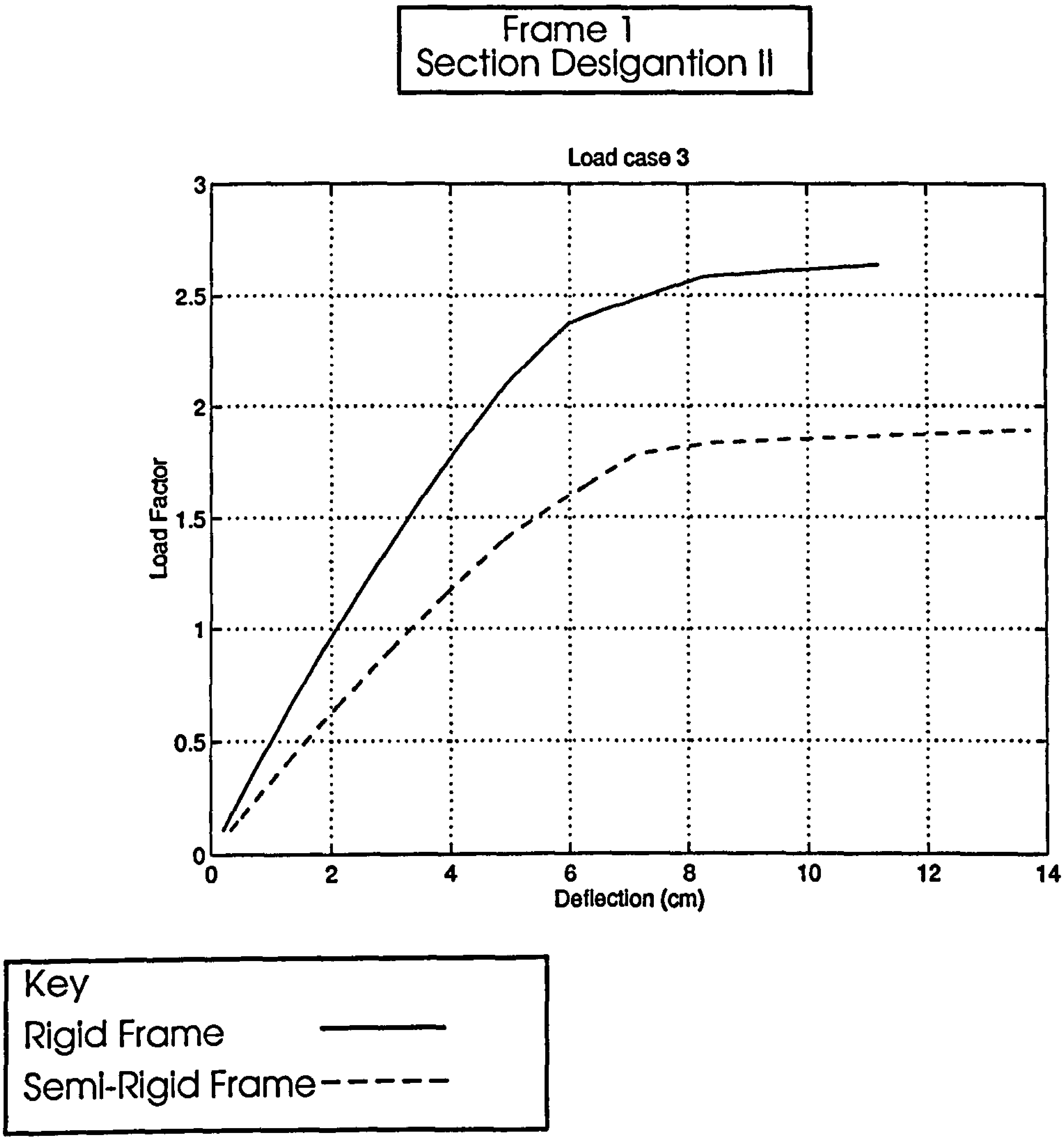


Fig. 3.14 (c) Load-deflection curves.

Chapter 4

4.0 Performance of flush end-plate joints connected to a column web.

4.1 Introduction.

This chapter examines the performance of steel flush end-plate joints connected to a column web by comparing experimental results due to Kim[4.1] with theoretical predictions for moment resistance and stiffness. The standard connection details used and their dimensions are shown in Fig. 1.26. The performance of such connections was further studied with a view to their possible use in wind-moment frames bending about the minor column axis. Experimental results for moment and stiffness are established from a series of twenty two tests presented in terms of graphs plotted as moment versus rotation[4.1]. For each of the tests, these M-Φ curves were plotted for deformations of the column web, the end plate, and the overall connection. To establish a predicted value for the moment resistance as limited by the column web, Gomes's formulae were adopted[4.2][4.3]; these have already has been described in Chapter 1.

The moment resistance of the connection is not only limited by the column web. It is also dependent on the moment resistance of the end-plate which was determined by adopting established design procedures[4.4]. With a known value of moment resistance due to both the column web and the end-plate, the theoretical moment resistance of the connection can be established. Good agreement was found between the experimental and theoretical results.

The study on Kim's results was further extended to compare the experimental and theoretical values of initial stiffness($S_{j,ini}$) for the connection. To establish the theoretical value, the initial stiffness of the column web and the end-plate plus bolts were calculated separately. This can be done by using finite element analysis to calculate the column web stiffness, and the component method described in the revised Annex J of Eurocode 3[4.5] to calculate the end plate plus bolt stiffness. The connection stiffness can then be predicted by combining the stiffness contributed by the column web and that due to the end plate plus bolts.

The study was then extended to the prediction of initial rotational stiffness for a range of combined column and beam sections which were selected from frames bending about the minor axis (described in Chapter 3). Sommer's method[4.6] was adopted to develop a prediction equation. Details of the procedure used are described later in this chapter.

The studies reported in this chapter conclude with an assessment of the suitability of these joints based up on the moment resistance required of the connections for frames analysed by the wind-moment method with bending about the minor axis. The design of such frames has been described in Chapter 3. The particular purpose of the study was to assess whether the required moment calculated from wind-moment analysis can be provided by the connections.

4.2 Test specimens and test rig used by Kim's experiments.

The test rig used by Kim for his experiments is shown in Fig. 1.25 . The arrangement was chosen so that deformation of column web would occur, representing the situation of an external column in a frame. The behaviour of such a connection was expected to be strongly influenced by deformation of the column web. Similar behaviour would also be expected in a double-sided configuration under unbalanced loading, such as that expected due to wind-moments. The test specimens are described in Table 1.1.

4.2.1 Test procedure.

A detailed description of the testing is available elsewhere[4.1]. A small load was applied and then removed, to check the performance of the rig. Significant load was then applied, sufficient to cause extensive inelastic deformation of the connection. To determine the complete response, each connection was later subjected to unloading, followed by reverse loading. The response of a joint in these phases may govern the buckling behaviour of the connected column. As this chapter describes the prediction of M_R and S_j in the first phase of loading, only this part of each connection's response is presented. The response of Test 1 is not given because this specimen was intended solely to prove the effectiveness of the test rig, and used an untypically thin end plate.

4.2.2 Experimental results.

The usual shape of the experimental M-Φ curve is shown in Fig. 4.1[4.1]. There is no plastic plateau because yielding of the end plate is followed by an increase of stiffness due to membrane effects. It is therefore necessary to determine a value of the moment resistance which corresponds to the substantial loss of stiffness observed in the tests prior to the onset of the membrane action. The experimental values of moment resistance M_R listed in Table 4.1 was determined by estimating when a "knee" formed in each of the M-Φ curves plotted (Fig. 4.1). By adopting this technique, the experimental values of M_R for the overall joint for the tests were established (Fig 4.2(a-u)). For curves which do not clearly show a linear region, an assumed straight line was drawn parallel to the unloading region traced from the exact M-Φ curves[4.1].

The initial stiffness $S_{j,ini}$ for the connection was established by drawing the best possible straight line (based on engineering judgement) along the assumed linear loading region of M-Φ curve for tests 2 to 13 (Fig 4.3(a-l) and the results are tabulated in Table 4.2. Only tests with a 152x152UC23 section were studied for the reasons explained in 4.4 below. The results show a slight increase in the stiffness as the thickness of the end plate increases. The stiffness also increases with the depth of the beam.

4.3 Prediction of moment resistance M_R

As explained earlier the prediction of moment resistance M_R accounting for the strength of the column web was performed by adopting Gomes's formulae[4.2][4.3]. In Gomes's formulae the variable parameters are the steel's yield strength, the thickness of the column web, the width of the beam, the column depth between fillets, the size of the bolt, the distance between bolts, and the lever arm of the connection. These determine the failure load value and thus the moment resistance. These variable parameters are included in the formulation described in Table 1.2. A computer program was developed to calculate the failure load, the moment resistance and the identification of failure mode for comparison with Kim's tests and the results are listed in Table 4.3. An example of calculating the failure load is shown in 4.3.1. Reference should be made to Table 1.2 in respect of the failure modes shown in Table 4.3.

In each of Kim's tests, it is assumed that the bottom row of bolts resists only vertical shear. Gomes' analysis does not deal with connections with more than one bolt-row in tension. To apply his analysis to Kim's tests 8-22, which have 2 rows assumed in tension, the limiting force given by the global mechanism is converted into two forces F_1 and F_2 as shown in Fig. 4.4 and described below. It can be seen from Table 4.3 that the global mechanism gives either the lowest failure load, or a value close to the lowest. For this reason no attempt has been made to apply Gomes' analysis for other failure modes to the case of more than one bolt row. It can be seen from Table 4.3 that F_1 is lower than the failure loads calculated from the other mechanisms, thereby justifying this approach. The force assigned for the lower bolt row, F_2, is assumed to correspond to a triangular distribution line (Fig.4.4).

The conversion of two bolt rows in tension from F_{eq}. into F_1 and F_2 is now described.

From the triangular distribution:

$$F_2 = \frac{h_2}{h_1} \times F_1 \tag{4.1}$$

For the same turning effect,

$$F_{eq.}h_1 = F_1 h_1 + F_2 h_2 \qquad (4.2)$$

Substitite equation (4.1) into equation (4.2),

$$F_{eq} = \frac{F_1 h_1 + \frac{F_1 h_2}{h_1} \times h_2}{h_1} \qquad (4.3)$$

This can be written as $F_1 = \dfrac{F_{eq.} \times h_1}{h_1 + \dfrac{h_2^2}{h_1}}$ (4.4)

and, $F_2 = F_{eq} - F_1$ (4.5)

where,

F_{eq} is calculated from Gomes' formulae for one bolt row,

h_1 is a lever arm of the connection measured from the first tension bolt row to the centre of compression in the beam flange,

h_2 is a lever arm of the connection measured from the second tension bolt row to the centre of compression in the beam flange.

The moment capacity governed by the column web was then calculated by multiplying F_1 with h_1 and F_2 with h_2.

Table 4.4 shows the results for the moment capacity of the end-plate calculated using procedures based on Eurocode 3[4.5] which have been described in detail in an SCI publication[4.4]. The strength of the end plate was taken from measured values in Kim's thesis[4.1] and the dimensions of each end plate are shown in Fig 1.26. For test specimens with 3 bolt rows, the procedures of calculating bolt forces F_1 and F_2 and the moment resistance are described in 4.3.2 below. Similar procedures to calculate bolt force F_1 were used for test specimens with 2 bolt rows.

Table 4.5 shows the results of a comparison between experimental values and theoretical values of moment resistance for the overall joint. The theoretical values of moment were the lesser of those from Tables 4.3 and 4.4. Comparison with the tests show that most of the results are slightly greater than the predicted values. This may due to a longer length of lever arm than that predicted, as the end plate (which projects below the beam flange) is the actual element that acts on the column web, not the beam flange. Therefore, the centre of compression may not be at the centre of the compression beam flange; it may occur somewhere between the compression beam flange and the tip of the end plate. Also determination of M_R from experimental M-Φ curves is not exact.

4.3.1 Calculation of moment resistance due to column web using Gomes formulae.

Example of calculation for 203x203UC46 and 254x102UB22 (Test No. 20 to 22).

Table 1.2, Fig. 1.26, Fig. 1.27 and Fig. 4.5 are referred to in the calculation.

Check for validity range:

Validity range: $b/L<0.8$ and $0.7 \leq h/(L-b) \leq 10$

$L = [H - 2 \times T_{cf} - 2(3r/4)] = [203.2 - 2\times11.0 - 2(3\times10.2/4)]=165.9$ mm

$d_m = (d_1 + d_2)/2 = (27.7 + 24)/2 = 25.85$ mm

$b = b_o + 0.9 \times d_m = 60 + 0.9 \times 25.85 = 83.27$ mm

$b/L = 83.27/165.9 = 0.50 < 0.8$ ok

$H_l = 209$ mm, $H_t = H_l + d_2/2 = 209 + 24/2 = 221$mm

$h = H_l - 0.1H_t/2 = 209 - 22.1/2 = 197.95$ mm

$h/(L - b) = 197.95/(165.9-83.27) = 2.4 < 10$ ok

Constants:

$$m_{pl} = \frac{1}{4} t_w^2 f_y = 0.25 \times 7.2^2 \times 319.3 = 4138\text{kN}$$

$$c = 0.9dm = 0.9 \times 25.85 = 23.27$$

$$\alpha = \frac{4}{1-b/L}\left(\pi\sqrt{1-b/L} + 2c/L\right) = \frac{4}{\left(1-83.77/165.9\right)}\left(\pi\sqrt{1-83.27/165.9} + \frac{2\times 23.27}{165.9}\right) = 20.06$$

$$\frac{(b+c)}{L} = \frac{(83.27+23.27)}{165.9} = 0.64 > 0.5$$

$$k = \begin{cases} 1 & \text{if } (b+c)/L \geq 0.5 \\ 0.7 + 0.6(b+c)/L & \text{if } (b+c)/L \leq 0.5 \end{cases}$$

Therefore k = 1.

Local failure: $F_{local} = m_{pl}\alpha k = 4138 \times 20.06 = 83.0kN$.

Global failure: $\left(\frac{h}{L-b}\right) = \frac{197.95}{(165.9-83.27)} = 2.4 > 1.0$

$$F_{global} = \begin{cases} m_{pl}\left(\frac{2b}{h} + \frac{\alpha k}{2} + \pi + \frac{2h}{L-b}\right) & \text{if } \frac{h}{L-b} \geq 1 \\ m_{pl}\left(\frac{2b}{h} + \frac{\alpha k}{2} + \pi + 2\right) & \text{if } \frac{h}{L-b} \leq 1 \end{cases}$$

$$F_{global} = 4138\left(\frac{2\times 83.27}{197.95} + \frac{20.06}{2} + \pi + \frac{2\times 197.95}{165.9-83.27}\right) = 77.81kN.$$

Punching shear failure:

case 1 : punching shear around n bolt heads:

$$F_{Q1} = n\pi d_m t_w \frac{f_y}{\sqrt{3}} = 2 \times \pi \times 25.85 \times 7.2 \times 319.3/\sqrt{3} = 215.6kN.$$

case 2 : punching shear around the rectangular area:

$$F_{Q1} = 2(b+c)t_w \frac{f_y}{\sqrt{3}} = 2(83.27+23.27)\times 7.2\times 319.3/\sqrt{3} = 282.8kN.$$

Combined flexural and punching shear failure:

$$F_{Q2} = \begin{cases} F_{local} & \text{if } t_w \le L/20 \\ 4m_{pl}\left[\frac{x\sqrt{L(a+x)}+c}{a+x} + \frac{2cx+x^2}{\sqrt{3}t_w(a+x)}\right] & \text{if } t_w > L/20 \end{cases}$$

L/20 = 165.9/20 = 8.3 > t_w= 7.3mm, therefore $F_{combine} = F_{local}$

Plastic failure load:

$$F_{pl} = \min\left(F_{locall}, F_{global}, F_{Q1}, F_{Q2}\right)$$

Therefore, $F_{pl} = F_{local}$ = 77.81 kN

4.3.2 Calculation of moment resistance due to end plate plus bolts according to EC3.

Procedures described in SCI publication[4.4] and Fig 1.26 are referred to in the calculation.

Calculation for 254x102UB22.

Connection geometry for the end plate:

$$m = \frac{g}{2} - \frac{t_b}{2} - 0.8s_{ww} = \frac{60}{2} - \frac{5.8}{2} - 0.8\times 8 = 20.7\text{mm}$$

$$e = \frac{b_p}{2} - \frac{g}{2} = \frac{120}{2} - \frac{60}{2} = 30\text{mm}$$

n is the smallest of e = 30mm (end plate) or 1.25m = 1.25 x 20.7 = 25.9mm (end plate).

Therefore, n = 25.9mm.

Potential resistance of bolts in tension zone for bolt row 1.

Calculation of the effective length of the T-stub for the bolt row below the beam flange of flush end plate is determined as follows:

For 6mm and 8mm thick end plate,

$g = 60$ mm $< 0.7 \times 101.6(B_b) = 71.12$mm

$T_b = 6.8 > 0.8 \times 6(t_p) = 4.8$mm, $T_b = 6.8 > 0.8 \times 8(t_p) = 6.4$mm,

Therefore, use $\mathrm{Min}\{\mathrm{Max}\{ii, iii\}, i\}$, where i,ii, and iii are yield line pattern as described in SCI publication[4.4].

Pattern (i) $L_{eff} = 2\pi m = 2 \times \pi \times 20.7 = 130.1$mm.

Pattern (ii) $L_{eff} = 4m + 1.25e = 4 \times 20.7 + 1.25 \times 30 = 120.3$mm.

Pattern (iii) $L_{eff} = \alpha m_1$, where α is obtained as follows:

$m_1 = m = 20.7$mm

$m_2 = (S_1 - H) - T_{bf} - 0.8 \times 10 = 60 - 6.8 - 8 = 45.2$mm.

$$\lambda_1 = \frac{m_1}{m_1 + e} = \frac{20.7}{20.75 + 30} = 0.41 \qquad \lambda_2 = \frac{m_2}{m_1 + e} = \frac{45.2}{20.7 + 30} = 0.89$$

$\alpha \;= 5.9$

$\alpha m = 5.9 \times 20.7 = 122.1$mm.

The maximum of pattern (ii) and (iii) $= 122.1$mm.

Therefore, the effective length $L_{eff}\, \mathrm{Min}\{\mathrm{Max}\{ii, iii\}, i\} = 122.1$mm.

For 10 mm thick end plate

$g = 60$ mm $< 0.7 \times 101.6(B_b) = 71.12$mm

$T_b = 6.8 < 0.8 \times 10(t_p) = 8.0$mm

Therefore, use $\text{Min}\left\{Max\left\{\left(\frac{ii+iii}{2}\right), ii\right\}, i\right\}$

For pattern $\left(\frac{\text{ii}+\text{iii}}{2}\right)=\left(\frac{120.3+122.1}{2}\right)=121.2\text{mm}$, $\text{Max}\left\{\left(\frac{\text{ii}+\text{iii}}{2}\right),\text{ii}\right\}=121.2\text{mm}$

$\text{Min}\left\{\text{Max}\left\{\left(\frac{\text{ii}+\text{iii}}{2}\right),\text{ii}\right\},\text{i}\right\}=121.2\text{mm}$

Calculation of M_p and P_r for each of the end plate according to its thickness.

$$M_p = \frac{L_{eff}\times t_p^2\times p_{yp}}{4} = \frac{121.2\times 6^2\times 315.7\times 10^{-3}}{4} = 344\text{kNmm (for 6mm thick end plate).}$$

$$M_p = \frac{L_{eff}\times t_p^2\times p_{yp}}{4} = \frac{121.2\times 8^2\times 266.6\times 10^{-3}}{4} = 517\text{kNmm (for 8mm thick end plate).}$$

$$M_p = \frac{L_{eff}\times t_p^2\times p_{yp}}{4} = \frac{121.2\times 10^2\times 256.4\times 10^{-3}}{4} = 777\text{kNmm} \qquad \text{(for 10mm thick end plate).}$$

For 6mm thick end plate

Mode 1: $P_r = \frac{4M_p}{m} = \frac{4\times 344}{20.7} = 67\text{kN.}$

Mode 2: $P_r = \frac{2M_p + n\sum P_t'}{m+n} = \frac{2\times 344+(25.9\times 2\times 88)}{20.7+25.9} = 113\text{kN}$

Mode 3: $P_r = \sum P_t' = 2\times 88 = 176\text{kN}$

For 8mm thick end plate

Mode 1: $P_r = \frac{4M_p}{m} = \frac{4\times 517}{20.7} = 100\text{kN.}$

Mode 2: $P_r = \frac{2M_p + n\sum P_t'}{m+n} = \frac{2\times 517+(25.9\times 2\times 88)}{20.7+25.9} = 120\text{kN}$

Mode 3: $P_r = \sum P_t' = 2\times 88 = 176\text{kN}$

For 10mm thick end plate

Mode 1: $P_r = \frac{4M_p}{m} = \frac{4 \times 777}{20.7} = 150\text{kN}.$

Mode 2: $P_r = \frac{2M_p + n\sum P_t'}{m+n} = \frac{2 \times 777 + (25.9 \times 2 \times 88)}{20.7 + 25.9} = 131\text{kN}$

Mode 3: $P_r = \sum P_t' = 2 \times 88 = 176\text{kN}$

Potential resistance of bolts in tension zone for bolt row 2.

For bolt row 2 alone.

Calculation of the effective length of the T-stub for the "free" bolt row with no stiffner or beam flange is determined as follows:

Use: $\text{Min}\{i, ii\} = \text{Min}\{130.1, 120.3\} = 120.3\text{mm}$

Therefore effective length L_{eff} is equal to 120.3mm which is slightly less than 121.2mm.

Therefore P_{r2} can be taken as the same as for P_{r1}.

Potential resistance of bolts in tension zone for bolt row 1 plus bolt row 2 combined.

Calculation of the effective length of the T-stub for the bolt row below the beam flange is taken as:

For $g > 0.7B_b$ or $T_b < 0.8t_p$,

use $\text{Max}\left\{\frac{ii}{2}, \frac{iii}{2}\right\} + \frac{p}{2}$ or else use $\text{Max}\left\{\frac{ii}{2}, \left(iii - \frac{ii}{2}\right)\right\} + \frac{p}{2}$ for each bolt row, where p is the distance between bolt row 1 and bolt row 2.

For 6mm and 8mm thick end plate

$g = 60 \text{ mm} < 0.7 \times 101.6(B_b) = 71.12\text{mm}$

$T_b = 6.8 > 0.8 \times 6(t_p) = 4.8\text{mm}$, $T_b = 6.8 > 0.8 \times 8(t_p) = 6.4\text{mm}$, For 2 bolt rows, L_{eff} for group = Max of $\frac{ii}{2}+\frac{ii}{2}+p$ or $\left(iii-\frac{ii}{2}\right)+\frac{ii}{2}+p$

L_{eff} for group = Max of $(\frac{120.3}{2}+\frac{120.3}{2}+60=180.3\text{mm}$ or $\left(122.1-\frac{120.3}{2}\right)+\frac{120.3}{2}+60=182.1\text{mm})$

= 182.1mm.

For 10mm thick end plate

g = 60 mm < 0.7 x 101.6(B_b) = 71.12mm and T_b = 6.8 < 0.8 x 10(t_p) = 8.0mm

Therefore, use $\text{Max}\left\{\frac{ii}{2},\frac{iii}{2}\right\}+\frac{p}{2}$ for each bolt row.

For 2 bolt rows, L_{eff} for group = Max of $\frac{ii}{2}+\frac{ii}{2}+p$ or $\frac{iii}{2}+\frac{ii}{2}+p$

L_{eff} for group = Max of $(\frac{120.3}{2}+\frac{120.3}{2}+60=180.3\text{mm}$ or $\left(\frac{122.1}{2}+\frac{120.3}{2}\right)+60=181.2\text{mm})$

= 181.2mm

Calculation of M_p and P_r for each of the end plate according to its thickness

$$M_p=\frac{L_{eff}\times t_p^2\times p_{yp}}{4}=\frac{182.1\times 6^2\times 315.7\times 10^{-3}}{4}=517\text{kN.mm}$$ (for 6mm thick end plate).

$$M_p=\frac{L_{eff}\times t_p^2\times p_{yp}}{4}=\frac{182.1\times 8^2\times 266.6\times 10^{-3}}{4}=777\text{kN.mm}$$ (for 8mm thick end plate).

$$M_p=\frac{L_{eff}\times t_p^2\times p_{yp}}{4}=\frac{181.2\times 10^2\times 256.4\times 10^{-3}}{4}=1161\text{kN.mm}$$ (for 10mm thick end plate).

For 6mm thick end plate

Mode 1: $P_r=\frac{4M_p}{m}=\frac{4\times 517}{20.7}=100\text{kN}.$

Mode 2: $P_r=\frac{2M_p+n\sum P_t'}{m+n}=\frac{2\times 517+(25.9\times 4\times 88)}{20.7+25.9}=220\text{kN}$

Mode 3: $P_r=\sum P_t'=4\times 88=352\text{kN}$

For 8mm thick end plate

Mode 1: $$P_r = \frac{4M_p}{m} = \frac{4 \times 777}{20.7} = 150\text{kN}.$$

Mode 2: $$P_r = \frac{2M_p + n\sum P_t'}{m+n} = \frac{2 \times 777 + (25.9 \times 4 \times 88)}{20.7 + 25.9} = 229\text{kN}$$

Mode 3: $$P_r = \sum P_t' = 4 \times 88 = 352\text{kN}$$

For 10mm thick end plate

Mode 1: $$P_r = \frac{4M_p}{m} = \frac{4 \times 1161}{20.7} = 224\text{kN}.$$

Mode 2: $$P_r = \frac{2M_p + n\sum P_t'}{m+n} = \frac{2 \times 1161 + (25.9 \times 4 \times 88)}{20.7 + 25.9} = 246\text{kN}$$

Mode 3: $$P_r = \sum P_t' = 4 \times 88 = 352\text{kN}$$

P_{r2} = Minimum of (P_{r2} alone or P_{r2+1} - P_{r1})

Therefore,

P_{r2} is equal to 100 - 67 = 33 kN (for 6mm thick end plate)

P_{r2} is equal to 150 - 100 = 50 kN (for 8mm thick end plate)

P_{r2} is equal to 224 - 131 = 93 kN (for 10mm thick end plate)

Modification of the plastic distribution of the bolt force[4.4] is not necessary as the thickness of the end plate is less than $\frac{d}{1.9} \times \sqrt{\frac{U_f}{P_{yp}}} = 14.6\text{mm}$

Calculation of moment capacity.

P_{r1} = 67 kN and P_{r2} = 33 kN (For 6mm thick end plate)

P_{r1} = 101 kN and P_{r2} = 48 kN (For 8mm thick end plate)

P_{r1} = 131 kN and P_{r2} = 93 kN (For 10mm thick end plate)

$h_1 = 190.6$ mm is lever arm taken from tension bolt row one to centre of compression beam's flange.

$h_2 = 130.6$ mm is lever arm taken from tension bolt row two to centre of compression beam's flange.

The moment capacity of the connection is taken as:

$$M_c = \sum(F_{ri} \times h_i)$$

$$M_c = \sum(F_{ri} \times h_i) = (67 \times 190.6 + 33 \times 130.6) \times 10^{-3} = 17.1 \mathrm{kN.m} \qquad \text{(for 6mm thick end plate)}$$

$$M_c = \sum(F_{ri} \times h_i) = (100 \times 190.6 + 48 \times 130.6) \times 10^{-3} = 25.3 \mathrm{kN.m} \qquad \text{(for 8mm thick end plate)}$$

$$M_c = \sum(F_{ri} \times h_i) = (131 \times 190.6 + 93 \times 130.6) \times 10^{-3} = 37.1 \mathrm{kN.m} \qquad \text{(for 10mm thick end plate)}$$

4.4. Prediction of initial rotational stiffness $S_{j,ini}$.

In this study, only tests with a 152x152UC23 section from Kim's tests[4.1] were investigated. The reason is to keep the column web thickness and distance between fillets constant but to vary the beam depth, beam width, and number of bolt rows (two and three bolt rows). As a result, the total deflection values which contribute to rotation and initial rotational stiffness for each of the tests can be compared, without additional variables arising from change of column section.

The approach is that of the component method[4.5], now applied to joints between steel H-columns and steel I-steel beams with flush end-plate connection to a column web (Fig 1.25(a)). The purpose of using this method is to enable the amount of stiffness contributed by each element in the connection to be accounted for. However, no component properties are available to predict the stiffness of column web[4.5]. Therefore, a method capable of calculating the deformation of a column web is needed to predict the stiffness. In this study, finite element analysis was adopted due to its availability and advantages as described earlier. The stiffness was calculated by applying a value of moment and then determining the rotation from the resulting deformation.

4.4.1 Prediction due to column web.

To enable a moment value due to the column web to be predicted, bolt forces need to be applied to the column web. As the bolts act in tension, the tension capacity value of the bolts is used. The bolt's force was taken as a tensile strength multiplied by the tensile stress area of the bolts ($F = p_t \times A_t$)[4.10], where for the bolts used by Kim, p_t is taken as 450N/mm^2 and A_t is equal to 157mm^2. Therefore, the force of one bolt is equal to 70.65kN. This bolt force was used for each of the tests with two bolt rows, assuming a linear-elastic behaviour of the connection. For three bolt rows, the tension force in the second bolt row was calculated by similar triangles. These bolt forces were then applied in the finite element analysis software, I-DEAS[4.11] to calculate the total deflection of column web. This software is capable of predicting the deformation of the face of a column web. The model used and the analysis method are described below. The moment value was calculated from the tension bolt forces multiplied by the lever arm taken as a distance from each of the tension bolts to the centre of the compression flange.

4.4.2 Model used

The software allows the column web to be modelled as a thin plate with specified dimensions of width, length, and thickness, specified properties of materials, and directions of bolt forces; constraint conditions are also specified. The column was modelled in such a way that the length of the panel was relatively long compared the width, so as to represent the "exact" situation. Both sides of the column web, attached in reality to a flange were fixed (encastre') and the far ends were freed as shown in Figure 4.6. This assumption is made due to a very stiff column flange compared with the column web on the deformed plane. Forces due to bolts in the tension zone are modelled as point loads acting outward from the plane of the column web. In the compression zone, equilibrium requires that the value of the force is equal to the total tension force. The compression force was assumed to be uniformly distributed on the column web across the width of the beam flange. This uniformly distributed force was applied to the web modelled as equally divided point loads applied at each of the nodes along the width of the beam. The number of nodes used depends on the size of mesh and is discussed later in the chapter.

4.4.3 Analysis method.

The aim of the analysis was to find a numerical value of the out-of-plane deformation of the surface of the column web. The modulus of elasticity was taken as 206.8 kN/mm^2 and, for Poisson's Ratio 0.29. During the deformation, the axis of the bolts remains unchanged. It was assumed that the deformation of the column web does not depend on the deformation of the end-plate.

The analysis started with the layout of the geometrical configuration of the connection based on the exact position of bolts, beam, and column web panel. The width of the column web is taken as the depth between the fillets of the column. The longer length of the panel is taken as five or six times the width of the column web. This is to ensure that the deflection was not influenced by the free ends. The mesh area was then created with emphasis on the development of nodes at the positions of bolt forces and the beam flanges. This is to ensure that the forces acting are at the correct positions. The number of nodes across the width of the compression beam flange is determined by the division of the mesh. In this study, the mesh size was divided into a "reasonable" size which was enough to develop a series of point loads to represent the distributed load (Fig. 4.6). Trial analyses showed that changing the mesh size did not change the deflection result significantly; it depends on the total force on the beam's flange which is independent of the mesh size. The mesh size adopted is shown in Fig. 4.7.

4.4.4 Deflection results.

Figure 4.7 shows typical deformation of the column web with tension and compression zones deformed in the opposite direction. Figure 4.8 shows that typical contours of deflection for the tension and compression zones are stretched to the boundary limits but do not cross the far end of the free edge. The maximum deflection values in both zones are recorded to calculate the total deflection of the column web. The total deflection of column web was calculated by adding together the deflection due to the tension from the bolts and deflection due to the compression from the beam's flange. The results of the total deflection were listed in Table 4.6. The results show that for three bolt rows the increase in deflection is about 50% above that for two bolt rows, for the same column and beam sizes. For two bolt rows, decrease in the size of the beam flange shows an increase in the total deflection. This is because the deflection contributed by compression from the beam flange is then concentrated more towards the centre of the panel. For three bolt rows, the total deflection increases as the depth of the beam increases. This is because the force in the second row of tension bolts is of more significance as the depth of the beam increases.

4.4.5 Development of total rotation and initial stiffness.

With known values of moment and rotation for the column web, it is possible to predict the initial stiffness. The total rotation (Φ) was calculated as the total deflection divided by a rotation length. For two bolt rows, the length of rotation is taken from the tension bolts to the centre of the compression flange of the beam (Fig. 4.9(a)). For three bolt rows the length is taken at the mid-distance between the first and second rows of tension bolts to the centre of compression beam flange (Fig 4.9(b)). This position was found from the finite element analyses to be that which gave the maximum deflection, to the accuracy of the mesh adopted. The moment, rotation, and calculated initial stiffness are listed in Table 4.7.

4.4.6 Prediction due to end plate plus bolts.

The initial stiffness of the end plate depends on the size of the beam and the thickness of the end plate; no column configuration was involved in the calculation. The stiffness of the end plate and bolts were calculated separately and then added together using formulae described in the revised Annex J[4.5]. Examples of the calculation for the initial stiffness of the end plate plus bolts are now described for two bolt rows and three bolt rows; the results are tabulated in Table 4.8.

The calculation of the initial stiffness is with reference to Fig 1.26 and the revised Annex J[4.5]. For 254x102UB22, Tp = 6 mm, and two bolt rows.

Stiffness coefficient for end-plate, single bolt-row in tension:

$$k_5 = \frac{0.85 l_{eff} t_p^{\,3}}{m^3}$$

where the values of m and l_{eff} were calculated as shown in 4.3.2.

l_{eff} = 122.1 for first bolt row, m = 20.7mm

$$k_5 = \frac{0.85 l_{eff} t_p^{\,3}}{m^3} = \frac{0.85 \times 122.1 \times 6^3}{20.7^3} = 2.53\text{mm}$$

Stiffness coefficient for bolts, single bolt row in tension:

$$k_7 = \frac{1.6 A_s}{L_b}, \text{ where}$$

A_s = 157 mm^2

L_b =5.8 + 6 + 3 + 0.5(10 + 13) = 26.3 mm

$$k_7 = \frac{1.6 A_s}{L_b} = \frac{1.6 \times 157}{26.3} = 9.55\text{mm}$$

$$S_{j(\text{end-plate+bolts})} = \frac{Ez^2}{\mu \sum_i \frac{1}{k_i}} = \frac{0.205 \times 190.6^2}{1 \times \left(\frac{1}{2.53} + \frac{1}{9.55} \right)} = 14.90 \text{kN.m/mrad.}$$

For 254x102UB22, Tp = 6 mm, and 6 bolts force.

Stiffness coefficient for end-plate, single bolt-row in tension:

$k_5 = \frac{0.85 l_{eff} t_p^{\ 3}}{m^3}$, where the values of m and l_{eff} were calculated as shown in 4.3.2.

l_{eff} = 122.1 for first bolt row,

l_{eff} = 120.3 for second bolt row, and m = 20.7mm

$$k_{5,1} = \frac{0.85 l_{eff} t_p^{\ 3}}{m^3} = \frac{0.85 \times 122.1 \times 6^3}{20.7^3} = 2.53 \text{mm}$$

$$k_{5,2} = \frac{0.85 l_{eff} t_p^{\ 3}}{m^3} = \frac{0.85 \times 120.3 \times 6^3}{20.7^3} = 2.49 \text{mm}$$

Stiffness coefficient for bolts, single bolt row in tension:

$k_7 = \frac{1.6 A_s}{L_b}$, where

$A_s = 157 \text{ mm}^2$

L_b =5.8 + 6 + 3 + 0.5(10 + 13) = 26.3 mm

$$k_7 = \frac{1.6 A_s}{L_b} = \frac{1.6 \times 157}{26.3} = 9.55 \text{mm}$$

Equivalent stiffness coefficient is calculated as follows:

$$k_{eff,r} = \frac{1}{\sum_i \frac{1}{k_{ir}}}$$

Bolt row 1: $\quad k_{eff,1} = \frac{1}{\frac{1}{2.53} + \frac{1}{9.55}} = 2.00 \text{mm}$

Bolt row 2: $k_{eff,1} = \dfrac{1}{\frac{1}{2.49}+\frac{1}{9.55}} = 1.98\text{mm}$

$$k_{eq} = \frac{\sum_r k_{eff,r} k_r}{z} = \frac{2.00 \times 190.6 + 1.98 \times 130.6}{160.6} = 3.98\text{mm}$$

$$S_{j(\text{end-plate+bolts})} = \frac{Ez^2}{\mu \sum_i \frac{1}{k_i}} = \frac{0.205 \times 160.6^2}{1 \times \left(\frac{1}{3.98}\right)} = 21.04\text{kN.m/mrad.}$$

4.4.7 Assemblage of column web stiffness and end-plate plus bolts stiffnesses.

The total initial stiffness of the overall connection was finally calculated by assembling the initial stiffness from the end plate and bolts with the initial stiffness from the column web using the following formula:- $\dfrac{1}{S_{j(\text{total})}} = \dfrac{1}{S_{j(\text{end-plate+bolts})}} + \dfrac{1}{S_{j(\text{columnweb})}}$. The approach to derive the formula is described below.

The components which comprise the column web and bolts, together with the end-plate are modelled as a spring as shown in Fig. 4.10. This spring represents the flexibility of each component which responds with displacement Δ to the load P applied (Fig 4.10). The approach for assembling the flexibility of each of the element is as follows:

$$P = S_j \times \Delta \tag{4.6}$$

$$\Delta = FP \tag{4.7}$$

$$\text{Total flexibility, } F_{total} = F_{col.web} + F_{endplate+bolts} \tag{4.8}$$

$$\Delta\ \text{total} = \Delta\ \text{col. web} + \Delta\ \text{end-plate+bolts} \tag{4.9}$$

Substitute equation (4.6) and (4.7) into (4.9)

Therefore, $\dfrac{P}{S_{j(total)}} = F_{col.web} \times P + F_{endplate+bolts} \times P$ (4.10)

As the value of P is a common factor in equation (4.10)

$$\frac{1}{S_{j(total)}} = F_{col.web} + F_{endplate+bolts} \tag{4.11}$$

$$F = 1/\,S_j \tag{4.12}$$

Substitute equation (4.12) into (4.11), $\dfrac{1}{S_{j(total)}} = \dfrac{1}{S_{j(end-plate+bolts))}} + \dfrac{1}{S_{j(columnweb)}}$ (4.13)

An example of assembling the initial stiffness of column web and the initial stiffness of end plate plus bolts is shown below:

$$\frac{1}{S_{j(total)}} = \frac{1}{S_{j(end-plate+bolts))}} + \frac{1}{S_{j(columnweb)}} = \frac{1}{4.11} + \frac{1}{0.34}$$

$$S_{j(total)} = 0.314\,\text{kN.m/mrad}$$

Table 4.9 shows the results of predicted total initial stiffness of the connection compared with the experimental values. The results show good agreement between predicted and experimental values. For predicted values, the result shows that the values are closely dependent on the initial stiffness of the column web. This is because the web is so much more flexible than the end plate, as shown in the calculations above.

4.5 Equation for prediction of initial stiffness.

In predicting the stiffness for connections connected to a column flange where frames bend about the major axis, an established method[4.5] is available. However, no such method has been established for predicting the stiffness for connections connected to a column web. Therefore, in this chapter, the author has developed an expression for predicting the initial stiffness of flush end plate connections connected to a column web in the form of a linear equation as shown in equation 1.20. The values of C_1, C_2 and K factors in the equation are to be evaluated in detail in this chapter.

4.5.1 Selection of columns and beam sections

In selecting the column and beam sections, it is important to ensure that the beam can be connected to the web of the column with flush end plate connections, and the problem of lack of space to fit is avoided. The minimum width of recommended end plates is 200 mm[4.4]. However, for column size UC203x203 due to lack of space the width is modified to 160 mm to fit the depth between the fillets of the column. To ensure that this width is acceptable, the distance between the bolt to the edge of the plate needs to be checked to satisfy the requirement according to BS5950 Part 1[4.10]. The required minimum edge is 1.25D where D is the diameter of the bolt. For the purpose of this study, flush end-plates with two rows of 20 mm diameter bolts were used with standard cross-centres[4.4](Fig. 2.3(a)). Therefore, the minimum distance provided by the end plate is ((160-90)/2)=35mm which is greater than 1.25D(25mm). The combined column and beam sections are listed in Table 4.10. The UC152x152 sections were not considered because of their very restricted depth between flanges.

4.5.2 Application of Sommer method

Sommer's method[4.6] as described in Chapter 1 was adopted to predict the initial stiffness from a single function of the general form $\Phi=\sum C_i(KM)^i$. The author modified this function to the form of $\Phi=C_1(KM)+C_2$ to suit the prediction of initial stiffness, which is, by definition, linear. The approach to formulating the equation started with the determination of the K factor where K has the form $K=\prod_{j=1}^{m} p_j^{a_j}$. The method to determine the K factor has been briefly described in 1.8.1, which shows the need to solve the a_j value first.

4.5.2.1 Determination of a_j.

In this study, the equation to find a_j as described in equation 1.23 shows the need to establish M and Φ values for each of the combined column and beam configurations. Gomes' formulae[4.2][4.3] were again adopted to evaluate the plastic failure load and the bolt forces and thus the moment of the connection, as has been described earlier. However, in this study Gomes' plastic failure load is to be considered simply as a convenient level of loading to apply in the elastic finite element analysis used to establish a linearly elastic M-Φ curve.

To evaluate a rotation value Φ, the bolt forces calculated from Gomes' formulae were applied to the finite element software to analyse the total deflection of column web and thus establish the rotation value as has been discussed earlier. The results of the rotation values for each combined column and beam configurations are listed in Table 4.11.

The determination of the a_j factor is related to linear-elastic M-Φ curves as shown in Fig. 1.28(b). For every pair of connections which differ by only one parameter, M-Φ curves are drawn as shown in Fig.1.28(b), using the finite element results. For example, M-Φ curves for data sets 3 and 9 in Table 4.11 have the column web thickness varied but with constant column depth, lever arm, and beam width. From these curves, at a particular rotation Φ, the value of M_1 (data set 3) and M_2 (data set 9) can be established. This particular rotation Φ was taken to be equal to 10 mrad, corresponding the value of moments M_1 and M_2 being listed in Table 4.11 as "factored moment". This results in developing the value of M_1 for data set 3 as 9.83kN.m ((10/15.84)x15.57) and M_2 for data set 9 as 17.12kN.m((10/12.57)x21.52). Parameters p_{j1} and p_{j2} are taken from the column web thickness as listed in Table 4.11 for these data sets. By using this approach, other possible values of a_j can be established by varying:

- column depth
- lever arm of beam
- width of beam.

These values of a_j were calculated from every possible combination of data listed in Table 4.11 and the proposed value of a_j for design was taken as an average value. Details of the calculation are listed in Table 4.12(a-d). The final values determined for a_j are the exponent value for K factor and listed as follows:

for column web thickness S, $a_j = -2.93$

for column depth between fillet D, $a_j = 2.52$

for lever arm distance H, $a_j = -1.80$

for width of beam B, $a_j = -0.57$

The effect of all the parameters was combined to give the standardisation constant "K".

$$K = S^{-2.93}.D^{2.52}.H^{-1.80}.B^{-0.57} \tag{4.14}$$

4.5.2.2 Determination of C_1 and C_2 constants

The constant values C_1 and C_2 as shown in equation 1.20 determine the gradient and the ordinate of the predicted equation. Rotation values in mrad from Table 4.11 were listed as values for the y-axis and KM values (where M is in kN.mm) were listed as values for the x-axis. In determining KM, M is the moment given in Table 4.11 which corresponds to the rotation values and K was determined from equation 4.14. The resulting plot is shown by circles on Fig. 4.11. These values were then curve-fitted using software (Matlab[4.12]) to evaluate the constants C_1 and C_2. This software is capable of developing the best possible straight line from the data given as x and y axes. With known values of C_1 and C_2 the prediction equation can thus be developed as $\Phi = (0.11(KM) + 1.28)\text{mrad}$. The accuracy of the standardisation procedures is shown in Figure. 4.11, which shows the rotation-moment points compared with the prediction using the constant values of C_1 and C_2. The established equation was found to produce accurate rotation-moment curves for flush end plate connections, within the ranges of variables used in the prediction. The non-zero rotation corresponding to zero moment indicates that the prediction equation is not applicable in the range close to the origin.

4.5.2.3 Verification of the equation.

To verify the equation, a set of combined column and beam of sections different from those used in the predition were selected. The approach of calculating moment and rotation was similar to the method used in 4.5.2.1. With a known value of moment, the rotation Φ calculated from equation 4.14 can be established. A comparison of rotation calculated from the equation and rotation calculated from finite element analysis to verify the predicted equation is shown in Table 4.13.

4.6 Application to wind-moment analysis.

For frames bending on minor axis as described in Chapter 3, the connections used (Fig. 3.10) were not directly connected to column web. In these connections the thick end plates and the extra welding needed to the column flanges may increase the cost of fabrication. To investigate if simpler connections can be adopted the actual moment capacity of connections to the column web needs to be studied. Therefore the aim of this study is to establish the moment capacity of minor-axis flush end plate connections and compare these with the required moment from wind moment analysis. Unbraced frames bending about the minor axis, described as "Section Designation II" in Chapter 3, were used in this chapter.

4.6.1 Moment resistance for flush end plate connection.

In this study, the moment capacity of the flush end-plate connection tabulated as "W1/20" (Fig 2.3(a)) in Chapter 2, with an end plate thickness of 12mm was investigated. With this thickness, the moment resistance as limited by the end plate needs to be considered only for webs of columns greater than 12mm, as demonstrated later in this chapter. The moment resistance due to the end plate can be calculated with the same procedure as described in 4.3.2. For an end plate of 12mm thick, bolts forces were calculated to be equal to 208kN. The moment resistance due to the end plate is established by multiplying the bolt force of 208kN with the lever arm of the connection measured from the tension bolt to the centre of compression in the beam flange as shown in Fig 4.12(a).

To establish the moment resistance as limited by the column web, Gomes's formulae were again adopted to find the failure load and thus establish the moment resistance, as has been explained earlier. The moment resistance of the connection is taken to be the lesser of the moment resistance due to the end plate and that due to the column web. This moment resistance is considered to be the moment provided by the connection for an external column.

4.6.2 Moment resistance for internal columns.

The approach to calculating the moment resistance provided by the connection to an internal column was based on that for an external column. This means that the internal column is first assumed to be an external column. For an internal column, wind moments acting as shown in Fig 4.12(b) result in twice the forces on the web as for an external column. This reduces the resistance on each side of the internal column to half of that when it is assumed to be external.

4.6.3 Analysis of results

The results for the required moment and the provided are tabulated in Table 4.14(a and b) with results in parentheses ( ) being the effective values for moment resistance of the connections due to the end plate. For Frames 1,2,7, and 8 (Table 4.14(a)), the designed beam of size 533x210UB82 would create problems with the width of the beam not fitting some of the designed columns. Only Frame 4 (Table 4.13(a)) has connections capable of providing a moment resistance to suit the required moment. The results show that most other wind-moment frames have some connections not capable to provide enough moment capacity to resist the moment required on both external and internal columns.

The moment resistance as limited by the column web increases as the depth of the column decreases. This is because the lower value of d/t increases the failure load calculated from Gomes fromulae, which increases the moment resistance. For column webs thicker than the end-plate (12mm) as in Frame 2 (Table 4.14(b)), the calculated moment due to the end plate is still greater than the moment due to the column web for an external column. This is because the failure load calculated using Gomes' formulae for thick column webs is governed by the flexural and punching shear mode, rather than the global mode. However, for an internal column, the moment due to the column web governed the moment capacity of the connection even for the web of 356x406UC287 (22.5mm thick and therefore almost twice the end plate thickness).

4.7 Conclusions

This chapter concludes that it is possible to predict the moment resistance (M_R) of flush end plate connections connected to a column web by adopting Gomes' formulae[4.2][4.3] for calculating moment resistance due to the column web and the existing component method of EC3[4.5] for calculating the moment resistance due to the end plate. The study also showed that the initial stiffness ($S_{j,ini}$) for the connections can be predicted. The initial stiffness of column web, developed from force-deformation relationships based on linear elastic finite element analysis is assembled with the initial stiffness of other relevent components[4.5]. The results of experimental moment(M_R) and stiffness($S_{j,ini}$) showed good agreement with predicted values.

The study also established a prediction equation for initial stiffness for flush end plate connections, a standard wind-moment connection[4.4] with two bolt rows.

Finally for flush end plate connections connected to column web, usually the moment provided by the connection is not capable of resisting the required moment calculated from wind moment analysis.

References

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Table 4.1: Experimental value M_R at "knee" for each of the test

Test Number	Beam Section	Column Section	End Plate Thickness (mm)	Column Web Thickness (mm)	Number of Bolt	Experiment Moment (Total) (kN.m)
2			6			8.0
3	152x89RSJ	152x152UC23	8	5.8	4	8.0
4			10			9.0
5			6			12.0
6	254x102UB22	152x152UC23	8	5.8	4	13.5
7			10			13.0
8			6			15.0
9	254x102UB22	152x152UC23	8	5.8	6	15.5
10			10			16.0
11			6			20.0
12	305x102UB25	152x152UC23	8	5.8	6	20.0
13			10			21.5
14			6			23.0
15	305x127UB42	203x203UC46	8	7.2	6	24.0
16			10			27.5
17			6			19.0
18	254x102UB22	152x152UC37	8	8.0	6	23.0
19			10			25.0
20			6			15.5
21	254x102UB22	203x203UC46	8	7.2	6	16.5
22			10			17.0

Table 4.2: Initial stiffness values from M-Φ curves of Kim's tests

Test Number	Beam Section	Column Section	End Plate Thickness (mm)	Column Web Thickness (mm)	Number of Bolt	Experimental Initial Stiffness (kN.m/mrad)
2			6			0.36
3	152x89RSJ	152x152UC23	8	5.8	4	0.38
4			10			0.43
5			6			1.04
6	254x102UB22	152x152UC23	8	5.8	4	1.13
7			10			1.18
8			6			0.94
9	254x102UB22	152x152UC23	8	5.8	6	0.94
10			10			1.00
11			6			1.60
12	305x102UB25	152x152UC23	8	5.8	6	1.54
13			10			1.65

Table 4.3 Theoretical values for moment in column web for Kim's tests

Test Number	Beam Section	Column Section	Column Web Thickness (mm)	Number of Bolt	F local (kN)	F global (kN)	F punching shear around n bolt heads (kN)	F punching shear around ☐ area (kN)	F flexural and punching shear (kN)	F plastic failure load (kN)	F equivalent (kN)	F_1 (kN)	F_2 (kN)	h_1 (mm)	h_2 (mm)	Theoretical Moment (kN.m)
2 3 4	152x 89x16	152x152x23	5.8	4	69.75	61.56	176.44	231.45	69.75	61.56	61.56	61.56	0	103.4	0	6.4
5 6 7	254x102x22	152x152x23	5.8	4	71.06	71.54	179.75	235.80	71.06	71.06	71.54	71.54	0	190.6	0	13.6
8 9 10	254x102x22	152x152x23	5.8	6	69.15	69.61	174.91	229.45	69.15	69.15	69.61	46.8	32.6	190.6	130.6	13.2
11 12 13	305x102x25	152x152x23	5.8	6	75.30	81.78	190.47	249.85	75.30	75.30	81.78	52	39.3	241.4	181.4	19.7
14 15 16	305x127x42	203x203x46	7.2	6	83.00	82.14	215.58	282.80	82.140	83.00	82.14	52.1	39.5	240.6	180.6	19.7
17 18 19	254x102x22	152x152x37	8.0	6	132.05	132.93	242.16	317.66	121.10	121.10	132.9	89.9	43.1	190.6	130.6	22.7
20 21 22	254x102x22	203x203x46	7.2	6	83.00	77.81	215.58	282.80	83.00	77.81	77.81	52.4	36.5	190.6	130.6	14.8

Table 4.4 Theoretical values for moment in end plate for Kim's tests.

Test Number	Beam Section	Column Section	End Plate Thickness (mm)	Number of Bolt	Mode 1 (kN)	Mode 2 (kN)	Mode 3 (kN)	F_1 (kN)	F_2 (kN)	h_1 (mm)	h_2 (mm)	Theoretical Moment (kN.m)
2			6		66	112		66	0			6.8
3	152x 89x16	152x152x23	8	4	98	120	176	98	0	103.4	0	10.1
4			10		148	131		131	0			13.5
5			6		67	113		67	0			12.8
6	254x102x22	152x152x23	8	4	101	120	176	101	0	190.6	0	19.3
7			10		151	131		131	0			25.0
8			6		67	113		67	33			17.1
9	254x102x22	152x152x23	8	6	101	120	176	101	48	190.6	130.6	25.5
10			10		151	131		131	93			37.1
11			6		67	113		67	33			22.2
12	305x102x25	152x152x23	8	6	101	120	176	101	48	241.4	181.4	33.0
13			10		151	131		131	33			48.5
14			6		71	114		71	35			23.4
15	305x127x42	203x203x46	8	6	107	122	176	107	52	240.6	180.6	35.1
16			10		101	134		134	106			51.4
17			6		67	113		67	33			17.1
18	254x102x22	152x152x37	8	6	101	120	176	101	48	190.6	130.6	25.5
19			10		151	131		131	93			37.1
20			6		67	113		67	33			17.1
21	254x102x22	203x203x46	8	6	101	120	176	101	48	190.6	130.6	25.5
22			10		151	131		131	93			37.1

Table 4.5 Theoretical and experimental values for moment in end plate, column web and overall connection for Kim's tests.

Test Number	Beam Section	Column Section	End Plate Thickness (mm)	Number of Bolt	Theoretical end plate moments (kN.m)	Theoretical column web moments (kN.m)	Theoretical overall joint moments (kN.m)	Experiment overall joint moments (kN.m)
2	152x 89x16	152x152x23	6	4	6.8	6.4	6.4	8.0
3			8		10.1		6.4	8.0
4			10		13.5		6.4	9.0
5	254x102x22	152x152x23	6	4	12.8	13.6	12.8	12.0
6			8		19.3		13.6	13.5
7			10		25.0		13.6	13.0
8	254x102x22	152x152x23	6	6	17.1	13.2	13.2	15.5
9			8		25.5		13.2	15.5
10			10		37.1		13.2	16.0
11	305x102x25	152x152x23	6	6	22.2	19.7	19.7	20.0
12			8		33.0		19.7	20.0
13			10		48.5		19.7	21.5
14	305x127x42	203x203x46	6	6	23.4	19.7	19.7	23.0
15			8		35.1		19.7	24.0
16			10		51.4		19.7	27.5
17	254x102x22	152x152x37	6	6	17.1	22.7	17.1	19.0
18			8		25.5		22.7	23.0
19			10		37.1		22.7	25.0
20	254x102x22	203x203x46	6	6	17.1	14.8	14.8	15.5
21			8		25.5		14.8	16.5
22			10		37.1		14.8	17.0

Table 4.6 Total deflection of column web from finite element analysis.

Test Number	Beam Section	Column Section	No. of Bolt	Col. web Thickness (mm)	Total Deflection (mm)
2 3 4	152x89RSJ	152x152UC23	4	5.8	4.47
5 6 7	254x102UB22	152x152UC23	4	5.8	4.33
8 9 10	254x102UB22	152x152UC23	6	5.8	6.48
11 12 13	305x102UB25	152x152UC23	6	5.8	6.66

Table 4.7: Predicted initial stiffness of column web.

Test No.	Beam Section	Column Section	No. of Bolt	Col. web Thickness (mm)	Total Deflection (mm)	Rotation Length (mm)	Rotation Φ in (mrad)	Moment (kN.m)	Initial Stiffness (kN.m/mrad)
2 3 4	152x89RSJ	152x152UC23	4	5.8	4.47	103.35	43.3	14.6	0.34
5 6 7	254x102UB22	152x152UC23	4	5.8	4.33	190.6	22.7	26.9	1.18
8 9 10	254x102UB22	152x152UC23	6	5.8	6.48	160.6	40.3	39.6	0.98
11 12 13	305x102UB25	152x152UC23	6	5.8	6.66	211.6	31.5	53.4	1.70

Table 4.8 Predicted initial stiffness of end plate plus bolts.

Test Number	Beam Section	Column Section	End Plate Thickness (mm)	Number of Bolt	Initial Stiffness (kN.m/mrad)
2			6		4.11
3	152x89RSJ	152x152UC23	8	4	7.47
4			10		10.28
5			6		14.85
6	254x102UB22	152x152UC23	8	4	26.56
7			10		36.07
8			6		21.04
9	254x102UB22	152x152UC23	8	6	37.69
10			10		50.97
11			6		36.49
12	305x102UB25	152x152UC23	8	6	65.30
13			10		88.31

Table 4.9 Predicted initial stiffness of the connection compared with experimental values.

Test Number	Beam Section	Column Section	End Plate Thickness (mm)	Number of Bolt	Predicted Initial Stiffness (kN.m/mrad)	Experimental Initial Stiffness (kN.m/mrad)
2			6		0.34	0.36
3	152x89RSJ	152x152UC23	8	4	0.33	0.38
4			10		0.33	0.43
5			6		1.09	1.04
6	254x102UB22	152x152UC23	8	4	1.13	1.13
7			10		1.14	1.18
8			6		0.94	0.94
9	254x102UB22	152x152UC23	8	6	0.96	0.94
10			10		0.96	1.00
11			6		1.62	1.60
12	305x102UB25	152x152UC23	8	6	1.66	1.54
13			10		1.67	1.65

Table 4.10. Column and beam sections used in predicting initial stiffness.

Data No.	Beam Section (UB)	Column Section (UC)	Data No.	Beam Section (UB)	Column Section (UC)
1	203x133x25	203x203x71	17	406x140x39	305x305x97
2	406x140x39	203x203x71	18	406x178x54	305x305x97
3	203x133x25	254x254x73	19	533x210x82	305x305x97
4	254x102x22	254x254x73	20	203x133x25	356x368x129
5	305x102x25	254x254x73	21	254x102x22	356x368x129
6	305x127x37	254x254x73	22	305x102x25	356x368x129
7	356x127x33	254x254x73	23	406x140x39	356x368x129
8	356x171x45	254x254x73	24	406x178x54	356x368x129
9	203x133x25	254x254x89	25	533x210x82	356x368x129
10	254x102x22	254x254x89	26	203x133x25	356x368x153
11	305x102x25	254x254x89	27	406x140x39	356x368x153
12	305x127x37	254x254x89	28	406x178x54	356x368x153
13	356x171x45	254x254x89	29	533x210x82	356x368x153
14	356x171x45	254x254x107	30	406x140x39	356x368x177
15	356x171x45	254x254x132	31	406x140x39	356x368x202
16	203x133x25	305x305x97			

Table 4.11. Data used to predict the equation of initial stiffness.

Data	Beam Section (UB)	Column Section (UC)	Factored Moment (kN.m)	Moment (kN.m)	Rotation (mrad)	Dist. bet. fillet (mm)	Col. web Thickness (mm)	Lever Arm (mm)	Beam Width (mm)
1	203x133x25	203x203x71	29.54	27.12	9.18	160.8	10	159.05	133.2
2	406x140x39	203x203x71	140.40	59.53	4.24	160.8	10	344.1	141.8
3	203x133x25	254x254x73	9.83	15.57	15.84	200.3	8.6	159.05	133.2
4	254x102x22	254x254x73	14.29	21.09	14.76	200.3	8.6	207.30	101.6
5	305x102x25	254x254x73	21.61	27.18	12.58	200.3	8.6	255.84	101.6
6	305x127x37	254x254x73	25.51	27.09	10.62	200.3	8.6	255.18	123.3
7	356x127x33	254x254x73	31.00	32.86	10.60	200.3	8.6	297.60	125.4
8	356x171x45	254x254x73	35.30	33.18	9.40	200.3	8.6	299.86	171.1
9	203x133x25	254x254x89	17.12	21.52	12.57	200.3	10.3	159.05	133.2
10	254x102x22	254x254x89	24.27	29.15	12.01	200.3	10.3	207.30	101.6
11	305x102x25	254x254x89	36.69	37.57	10.24	200.3	10.3	255.84	101.6
12	305x127x37	254x254x89	43.45	37.45	8.62	200.3	10.3	255.18	123.3
13	356x171x45	254x254x89	59.79	45.86	7.67	200.3	10.3	299.86	171.1
14	356x171x45	254x254x107	112.45	67.13	5.97	200.3	12.8	299.86	171.1
15	356x171x45	254x254x132	187.18	95.46	5.10	200.3	15.3	299.86	171.1
16	203x133x25	305x305x97	9.39	17.65	18.80	246.7	9.9	159.05	133..2
17	406x140x39	305x305x97	36.15	42.66	11.80	246.7	9.9	344.10	141.8
18	406x178x54	305x305x97	41.38	43.33	10.47	246.7	9.9	348.45	177.7
19	533x210x82	305x305x97	72.60	59.75	8.23	246.7	9.9	467.90	208.8
20	203x133x25	356x368x129	7.47	16.87	22.57	290.2	10.4	159.05	133.2
21	254x102x22	356x368x129	10.44	22.81	21.85	290.2	10.4	207.30	101.6
22	305x102x25	356x368x129	15.88	28.75	18.10	290.2	10.4	255.84	101.6
23	406x140x39	356x368x129	28.16	40.52	14.39	290.2	10.4	344.10	141.8
24	406x178x54	356x368x129	30.31	41.13	13.57	290.2	10.4	348.45	177.7
25	533x210x82	356x368x129	55.55	58.66	10.56	290.2	10.4	467.90	208.8
26	203x133x25	356x368x153	12.27	24.53	20.00	290.2	12.3	159.05	133.2
27	406x140x39	356x368x153	46.22	56.67	12.26	290.2	12.3	344.10	141.8
28	406x178x54	356x368x153	49.77	57.53	11.56	290.2	12.3	348.45	177.7
29	533x210x82	356x368x153	90.96	82.05	9.02	290.2	12.3	467.90	208.8
30	406x140x39	356x368x177	73.41	77.67	10.58	290.2	14.4	344.10	141.8
31	406x140x39	356x368x202	109.30	101.98	9.33	290.2	16.6	344.10	141.8

Table 4.12 Determination of exponent values a_j.

Table 4.12(a) Vary column web thickness only (column depth, lever arm, and beam width are constant)

Data combined	(M_1/M_2)	$\log_{10}(M_1/M_2)$	(P_{j2}/P_{j1})	$\log_{10}(P_{j2}/P_{j1})$	a_j
3&9	0.574	-0.241	1.198	0.078	-3.08
4&10	0.589	-0.230	1.198	0.078	-2.94
5&11	0.589	-0.230	1.198	0.078	-2.94
6&12	0.587	-0.231	1.198	0.078	-2.95
20&26	0.609	-0.215	1.183	0.073	-2.95
23&27	0.609	-0.215	1.183	0.073	-2.95
24&28	0.609	-0.215	1.183	0.073	-2.96
25&29	0.611	-0.214	1.183	0.073	-2.94
13&14	0.532	-0.274	1.243	0.094	-2.91
13&15	0.319	-0.496	1.485	0.172	-2.88
14&15	0.601	-0.221	1.195	0.077	-2.86
27&30	0.630	-0.201	1.175	0.070	-2.88
27&31	0.423	-0.374	1.341	0.128	-2.93
30&31	0.672	-0.173	1.146	0.059	-2.92
				Average a_j	-2.93

Table 4.12(b) Vary column depth only (column web thickness, lever arm, and beam width are constant)

Data combined	(M_1/M_2)	$\log_{10}(M_1/M_2)$	(P_{j2}/P_{j1})	$\log_{10}(P_{j2}/P_{j1})$	a_j
1&16	3.147	0.498	1.534	0.186	2.68
2&17	3.884	0.589	1.533	0.186	3.17
9&20	2.290	0.360	1.449	0.161	2.24
10&21	2.325	0.366	1.449	0.161	2.28
11&22	2.310	0.364	1.449	0.161	2.26
				Average a_j	2.52

Table 4.12(c) Vary lever arm only (column web thickness, column depth, and beam width are constant)

Data combined	(M_1/M_2)	$\log_{10}(M_1/M_2)$	(P_{j2}/P_{j1})	$\log_{10}(P_{j2}/P_{j1})$	a_j
6&7	0.823	-0.085	1.166	0.067	-1.27
4&5	0.661	-0.180	1.234	0.091	-1.97
10&11	0.662	-0.179	1.234	0.091	-1.96
21&22	0.657	-0.182	1.234	0.091	-2.00
				Average a_j	-1.80

Table 4.12(d) Vary width of beam only (column web thickness, lever arm, and column depth are constant)

Data combined	(M_1/M_2)	$\log_{10}(M_1/M_2)$	(P_{j2}/P_{j1})	$\log_{10}(P_{j2}/P_{j1})$	a_j
5&6	0.847	-0.072	1.214	0.084	-0.86
7&8	0.878	-0.056	1.364	0.135	-0.42
11&12	0.844	-0.073	1.214	0.084	-0.87
17&18	0.874	-0.059	1.253	0.098	-0.60
23&24	0.929	-0.032	1.253	0.098	-0.33
27&28	0.929	-0.032	1.253	0.098	-0.33
				Average a_j	-0.57

Table 4.13. Data used to predict the equation of initial stiffness.

Data	Beam Section (UB)	Column Section (UC)	Col. web Thickness (mm)	Dist. bet. fillet (mm)	Lever Arm (mm)	Beam Width (mm)	Moment (kN.m)	Rot. from finite element (mrad)	Rot from predicted equation (mrad)
1	305x102x33	254x254x73	8.6	200.3	263.82	102.4	27.40	12.30	12.17
2	305x102x33	254x254x89	10.3	200.3	263.82	102.4	37.77	9.96	10.13
3	406x140x46	305x305x97	9.9	246.7	348.94	142.4	43.16	11.30	10.90
4	406x140x46	356x368x129	10.4	290.2	348.94	142.4	41.02	13.89	13.19
5	406x178x60	356x368x129	10.4	290.2	352.83	177.8	41.63	12.32	11.72
6	406x178x60	356x368x153	12.3	290.2	352.83	177.8	58.03	10.81	10.18

Table 4.14(a) Wind-moment design considering minimum wind in conjunction with maximum gravity load
Predicted moment for internal and external columns.

Basic Frame Type	Frame Identification	Width of Bay (m)	Section Designation (II)					Bending Moment Required (kNm)		Bending Moment Provided (kNm)			
			Universal Beam			Universal Columns				Floor		Roof	
			Floor	Roof		External	Internal	Floor	Roof	External Column	Internal Column	External Column	Internal Column
2 Storey 2 Bay	Frame 1	6.0 (composite floor at 3m span)	1st 533x210x82	356x171x45	Upto 2nd Storey	254x254x73	305x305x97	1st 61	27	Not Fit	29.88	33.18	18.03
4 Storey 2 Bay	Frame 2		1st 533x210x82 2nd 533x210x82 3rd 533x210x82	356x171x45	Upto 2nd Storey 2nd-4th Storey	305x305x97 254x254x73	356x368x129 254x254x89	1st 108 2nd 86 3rd 69	29	59.75 59.75 No Fit	29.33 29.33 No Fit	33.18	22.93
4 Storey 4 Bay	Frame 7		1st 533x210x82 2nd 533x210x82 3rd 533x210x82	356x171x45	Upto 2nd Storey 2nd-4th	305x305x97 254x254x73	356x368x129 254x254x89	1st 79 2nd 68 3rd 59	27	59.75 59.75 No Fit	29.33 29.33 No Fit	33.18	22.93
4 Storey 6 Bay	Frame 8		1st 533x210x82 2nd 533x210x82 3rd 533x210x82	356x171x45	Upto 2nd Storey 2nd-4th	305x305x118 254x254x73	356x368x129 254x254x89	1st 79 2nd 68 3rd 59	27	84.60 84.60 No Fit	29.33 29.33 No Fit	33.18	22.93
2 Storey 2 Bay	Frame 4	6.0 (precast floor at 6m span)	1st 203x133x25	203x133x25	Upto 2nd Storey	203x203x71	203x203x71	1st 24	7	27.12	13.56	27.12	13.56
4 Storey 2 Bay	Frame 5		1st 305x102x25 2nd 203x133x25 3rd 203x133x25	203x133x25	Upto 2nd Storey 2nd-4th Storey	305x305x97 203x203x60	356x368x129 254x254x73	1st 79 2nd 54 3rd 33	11	29.88 17.65 24.87	14.38 8.39 7.78	24.87	7.78
4 Storey 4 Bay	Frame 9		1st 305x102x25 2nd 254x102x25 3rd 203x133x25	203x133x25	Upto 2nd Storey 2nd-4th Storey	305x305x97 203x203x60	356x368x129 254x254x89	1st 39 2nd 27 3rd 16	5	29.88 23.90 24.87	14.38 8.39 7.78	24.87	7.78
4 Storey 6 Bay	Frame 10		1st 254x102x25 2nd 203x133x25 3rd 203x133x25	203x133x25	Upto 2nd Storey 2nd-4th Storey	305x305x97 203x203x60	356x368x129 254x254x89	1st 26 2nd 18 3rd 11	4	23.90 17.65 24.87	11.58 8.44 10.76	24.87	7.78

Table 4.14(b) Wind-moment design considering maximum wind in conjunction with minimum gravity load
Predicted moment for internal and external columns

Basic Frame Type	Frame Identification	Width of Bay (m)	Section Designation (II) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Bending Moment Required (kNm) Floor	Bending Moment Required (kNm) Roof	Bending Moment Provided (kNm) Floor External Column	Floor Internal Column	Roof External Column	Roof Internal Column
2 Storey 2 Bay	Frame 1	6.0 (composite floor at 3m span)	1st 457x152x60	356x171x45	Upto 2nd Storey	305x305x97	356x368x129	1st 120	45	48.30	25.41	36.06	17.23
4 Storey 2 Bay	Frame 2		1st 610x229x101 2nd 533x210x92 3rd 457x152x74	356x171x45	Upto 2nd Storey 2nd-4th Storey	356x368x153 356x368x129	356x406x287 356x368x202	1st 331 2nd 221 3rd 134	52	94 (111) 82 (96) 49.35	152.68(111) 133.96(96) 61.59	34.46	43.37
4 Storey 4 Bay	Frame 7		1st 533x210x82 2nd 457x152x67 3rd 457x152x60	356x171x45	Upto 2nd Storey 2nd-4th	305x305x118 305x305x97	356x368x153 356x368x129	1st 172 2nd 117 3rd 76	33	84.60 72.52 48.30	41.03(95) 34.12(80) 25.41	36.06	17.23
4 Storey 6 Bay	Frame 8		1st 457x152x60 2nd 457x152x60 3rd 457x152x60	356x171x45	Upto 2nd Storey 2nd-4th	305x305x97 254x254x73	356x368x129 305x305x118	1st 119 2nd 84 3rd 59	28	48.30 48.30 44.42	25.41 25.41 35.97	33.18	25.53
2 Storey 2 Bay	Frame 4	6.0 (precast floor at 6m span)	1st 406x140x46	305x102x33	Upto 2nd Storey	305x305x97	356x368x129	1st 107	30	43.42	20.61	30.87	14.84
4 Storey 2 Bay	Frame 5		1st 610x229x101 2nd 533x210x92 3rd 457x152x74	305x102x33	Upto 2nd Storey 2nd-4th Storey	356x368x153 356x368x129	356x406x287 356x368x202	1st 318 2nd 207 3rd 120	38	94 (111) 82 (96) 49.35	152.68(111) 133.96(96) 61.59	29.67	37.34
4 Storey 4 Bay	Frame 9		1st 533x210x82 2nd 457x152x67 3rd 406x140x39	254x102x25	Upto 2nd Storey 2nd-4th Storey	305x305x118 305x305x97	356x368x153 356x368x129	1st 159 2nd 104 3rd 60	19	84.60 72.52 42.66	41.03 34.12 20.26	23.90	11.59
4 Storey 6 Bay	Frame 10		1st 457x152x52 2nd 406x140x46 3rd 356x127x39	203x133x25	Upto 2nd Storey 2nd-4th Storey	305x305x97 254x254x73	356x368x129 305x305x118	1st 106 2nd 69 3rd 40	13	50.23 43.42 33.45	23.8 20.61 25.72	15.57	12.49

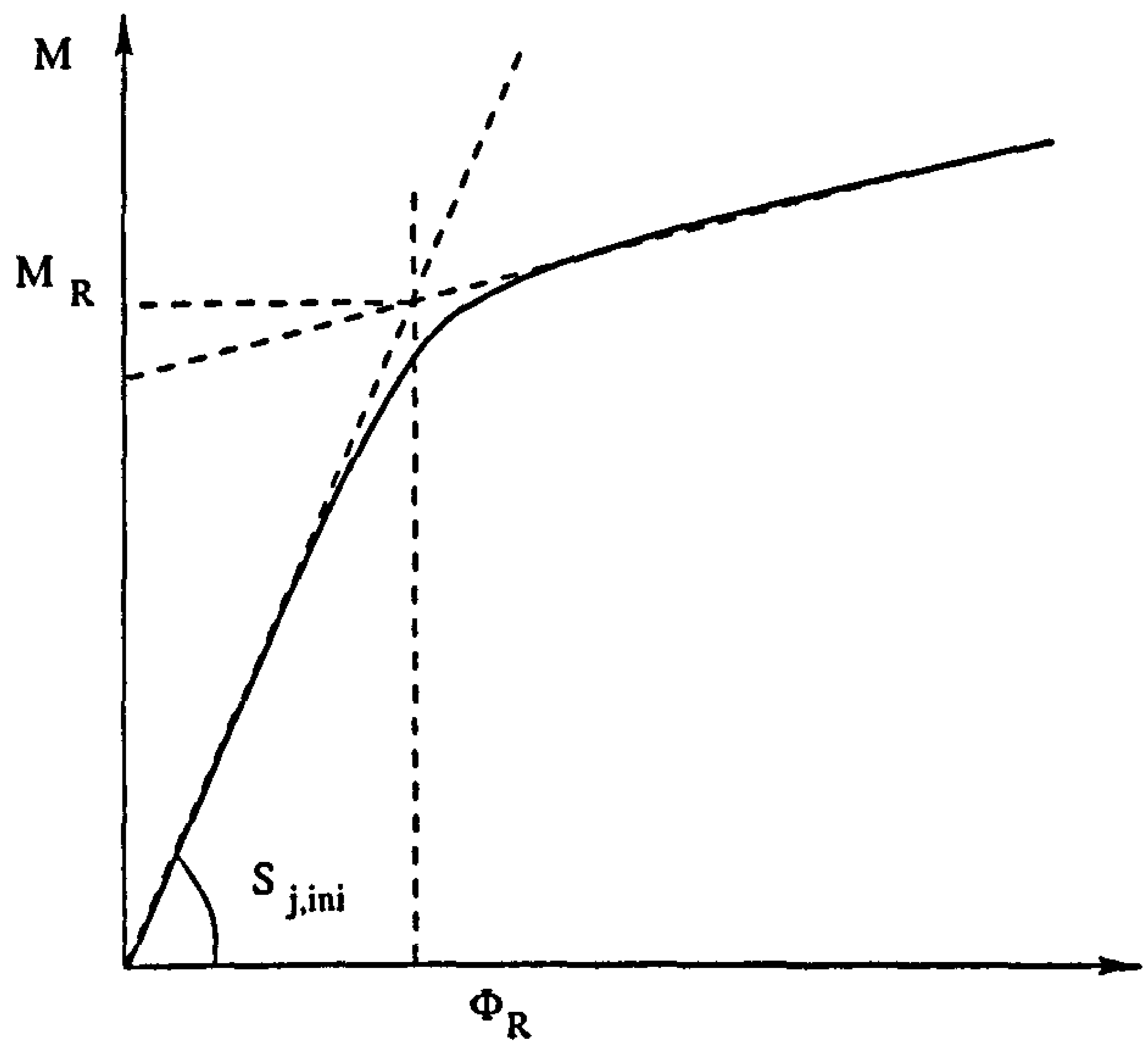

Fig. 4.1: Formation of "knee" to evaluate moment M and $S_{j,ini}$

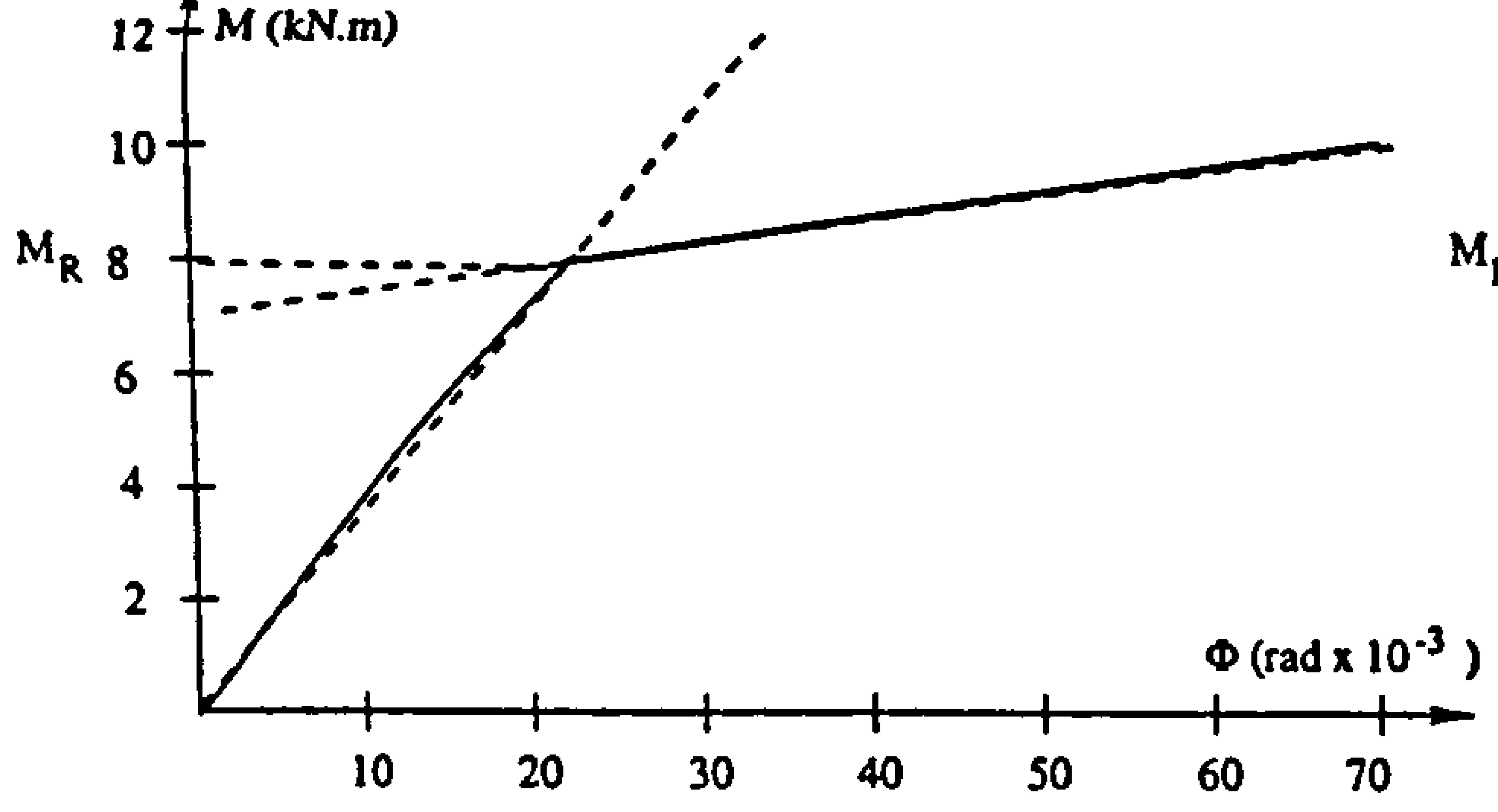

Fig. 4.2(a). Test No. 2 (M_R = 8.0 kN.m)

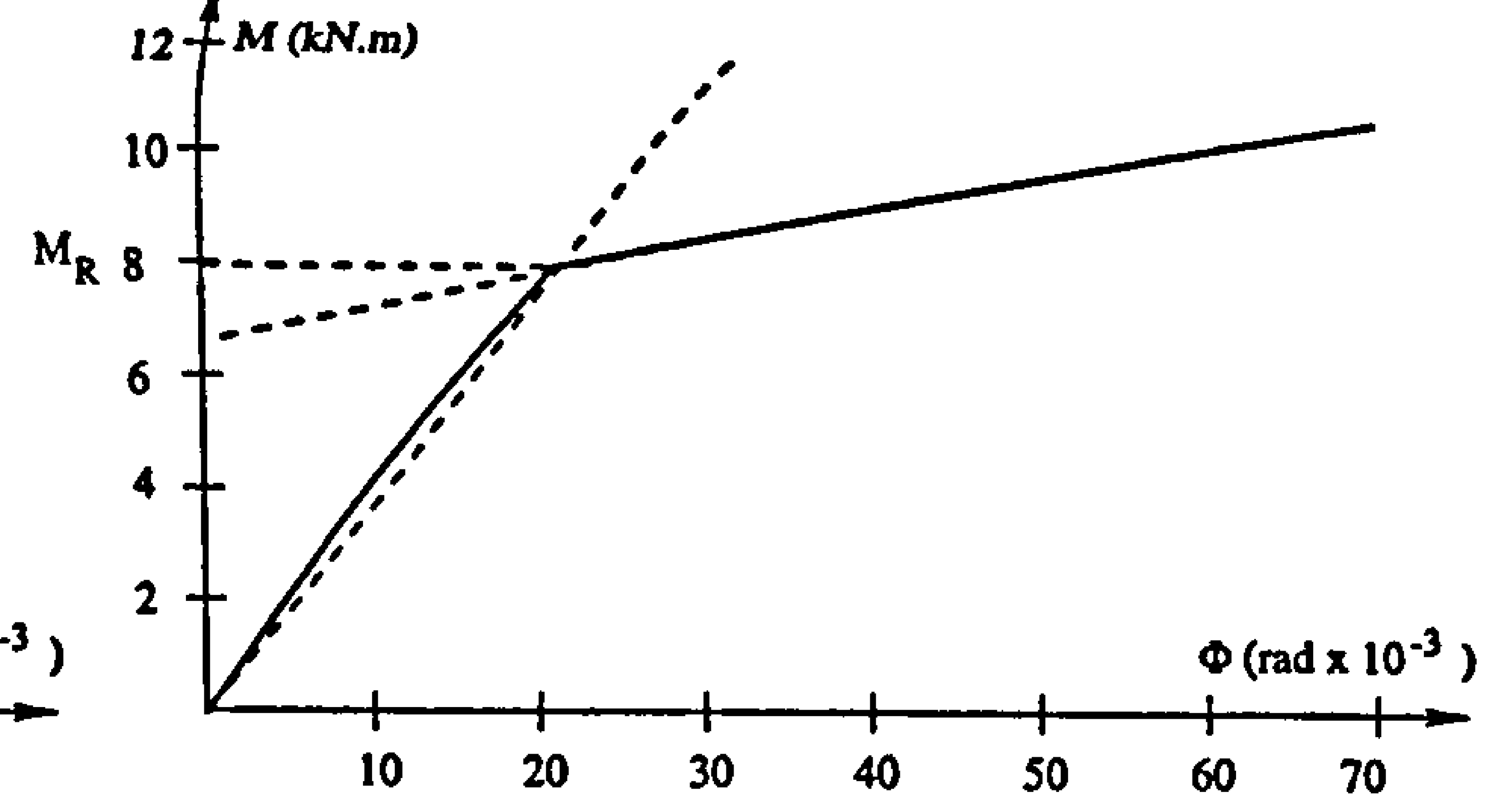

Fig. 4.2(b). Test No. 3 (M_R = 8.0 kN.m)

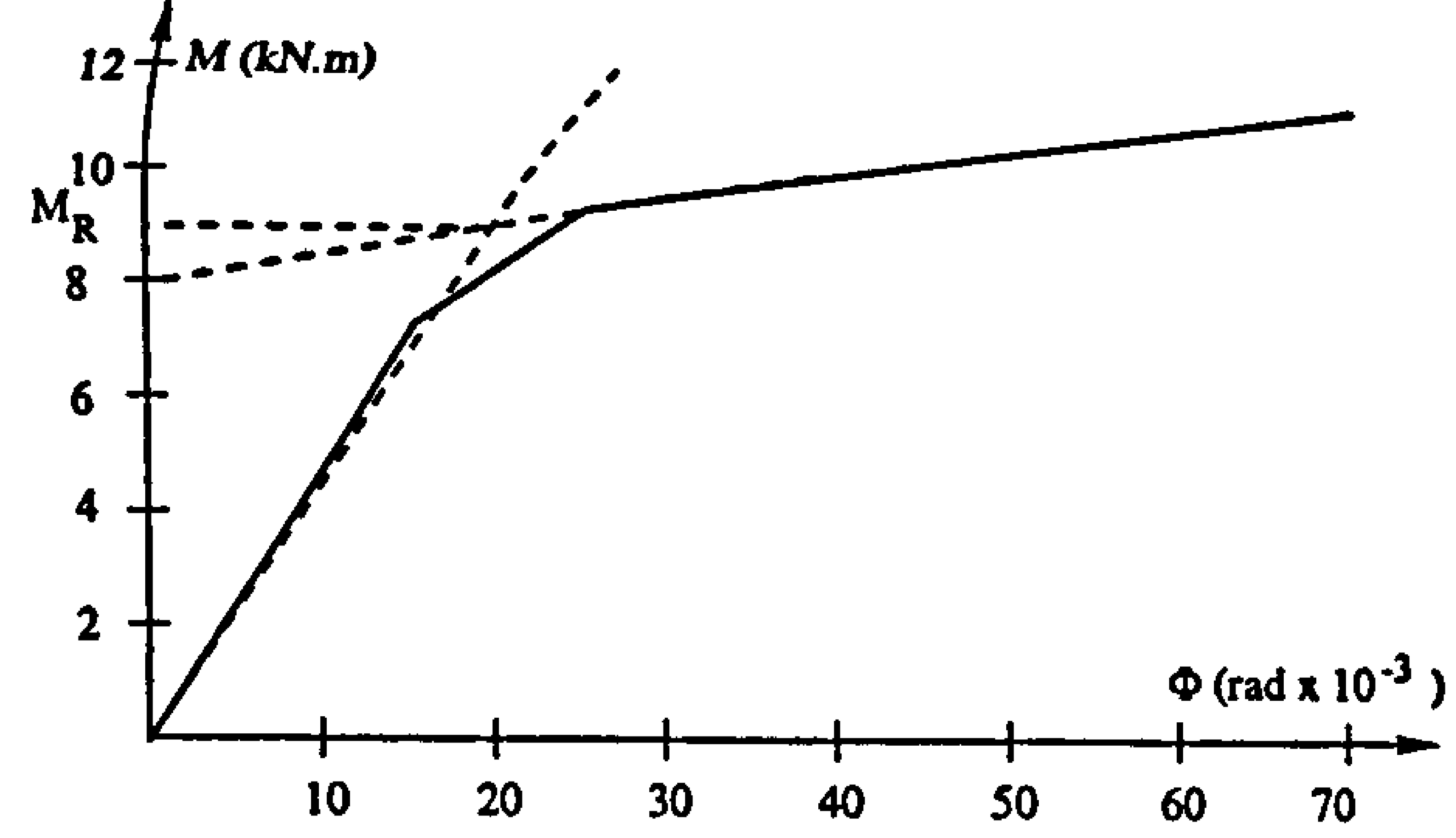

Fig. 4.2(c). Test No. 4 (M_R = 9.0 kN.m)

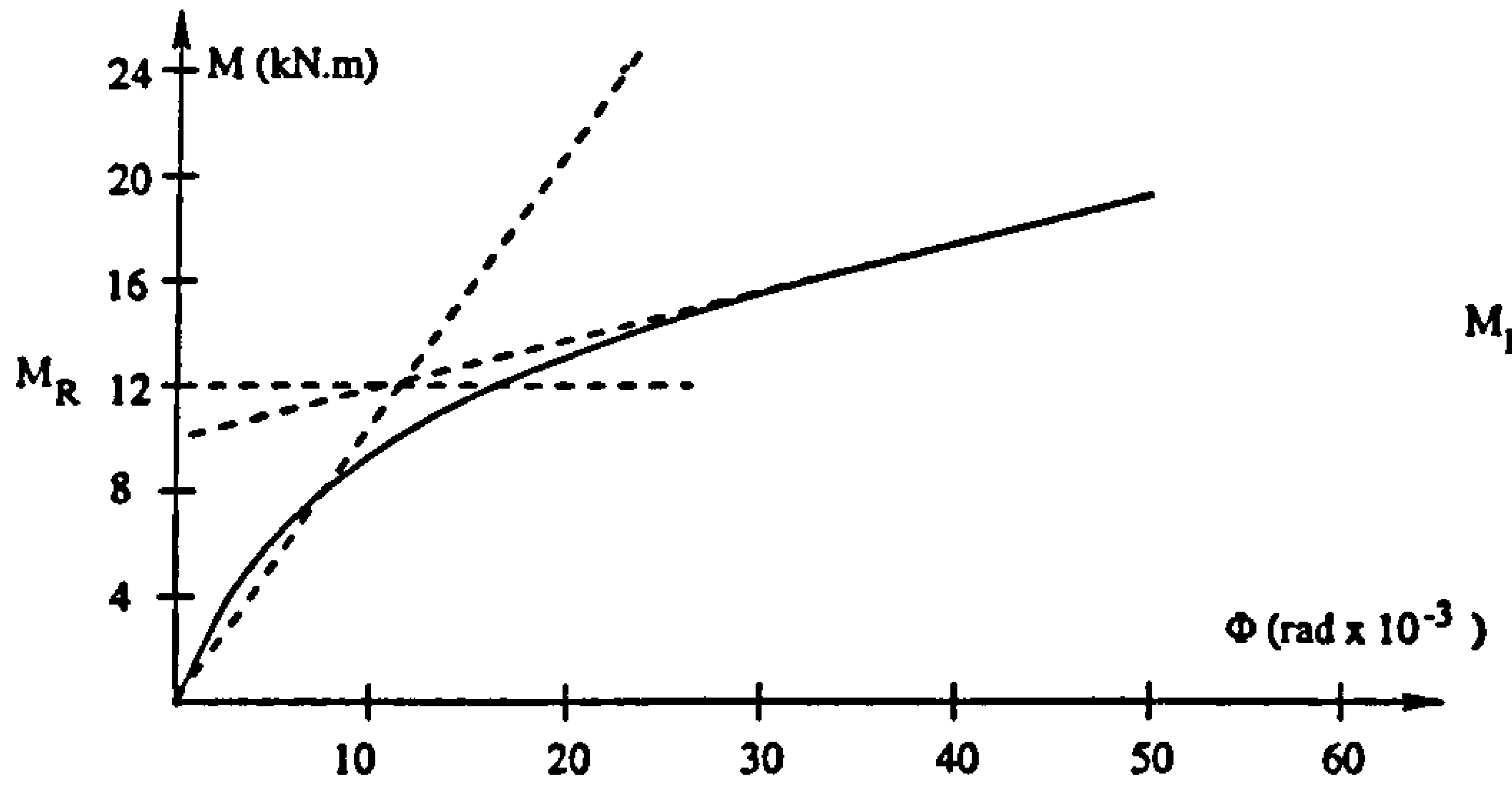

Fig. 4.2(d). Test No. 5 (M_R = 12.0 kN.m)

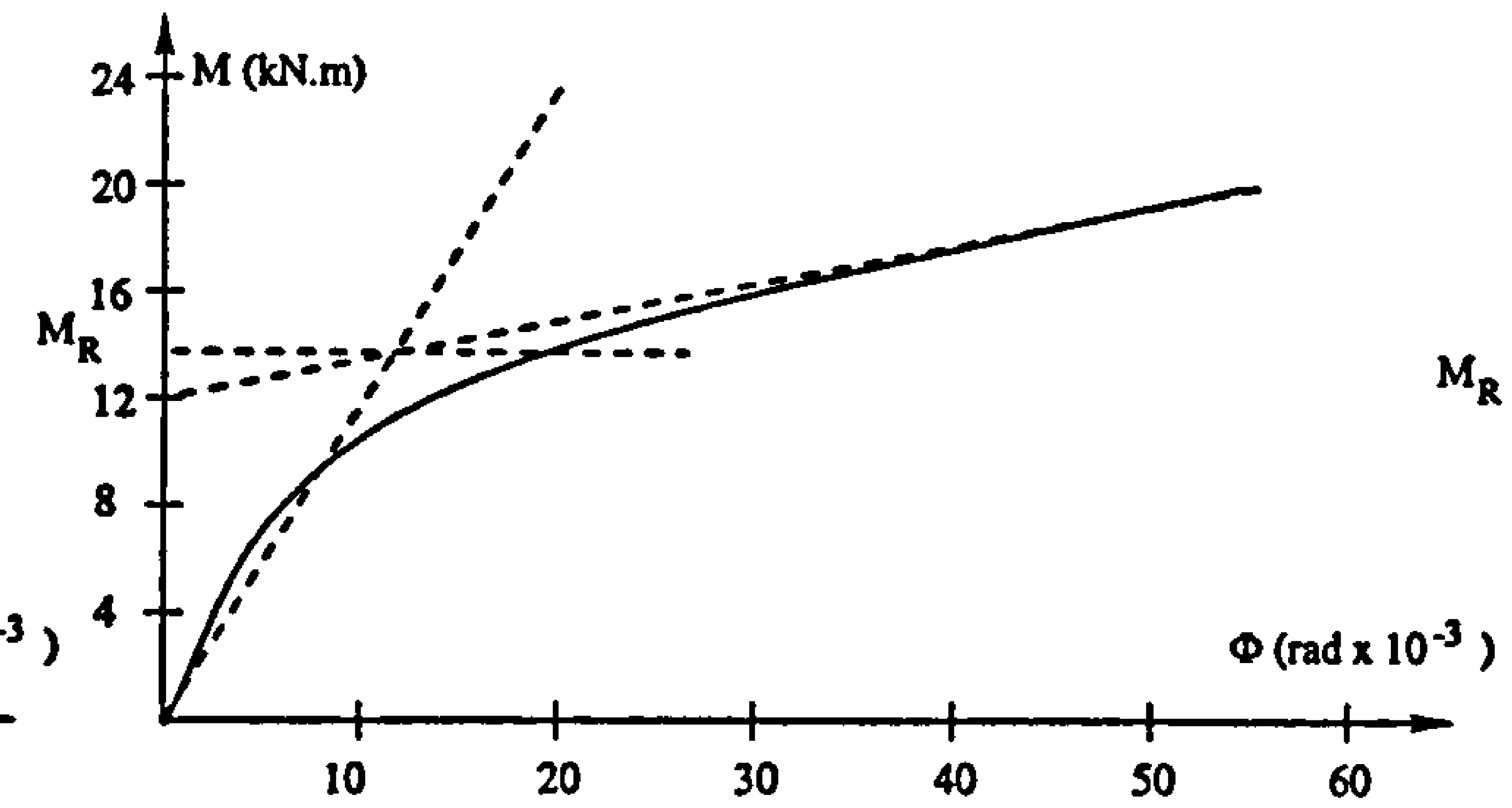

Fig. 4.2(e). Test No. 6 (M_R = 13.5 kN.m)

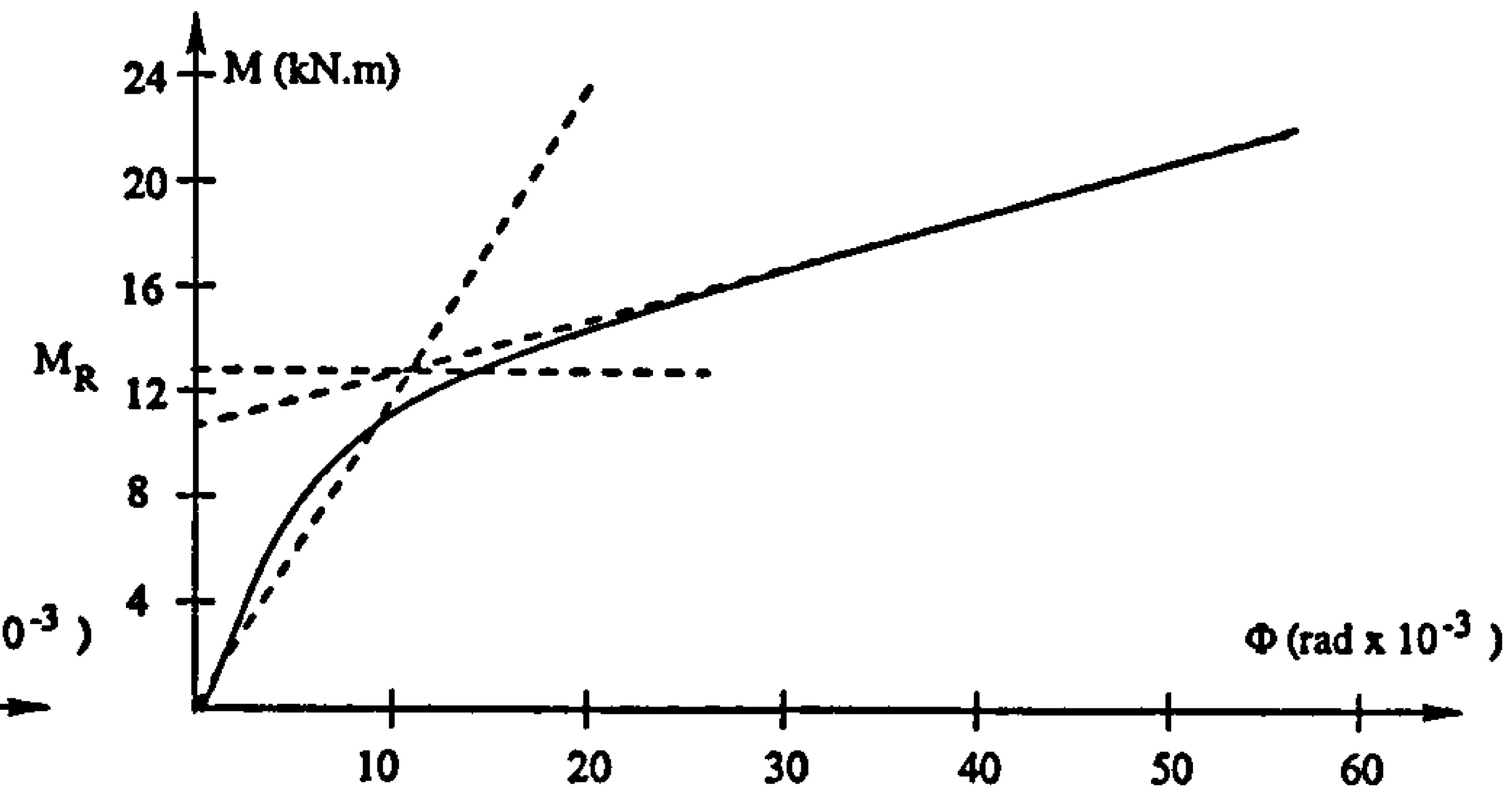

Fig. 4.2(f). Test No. 7 (M_R = 13.0 kN.m)

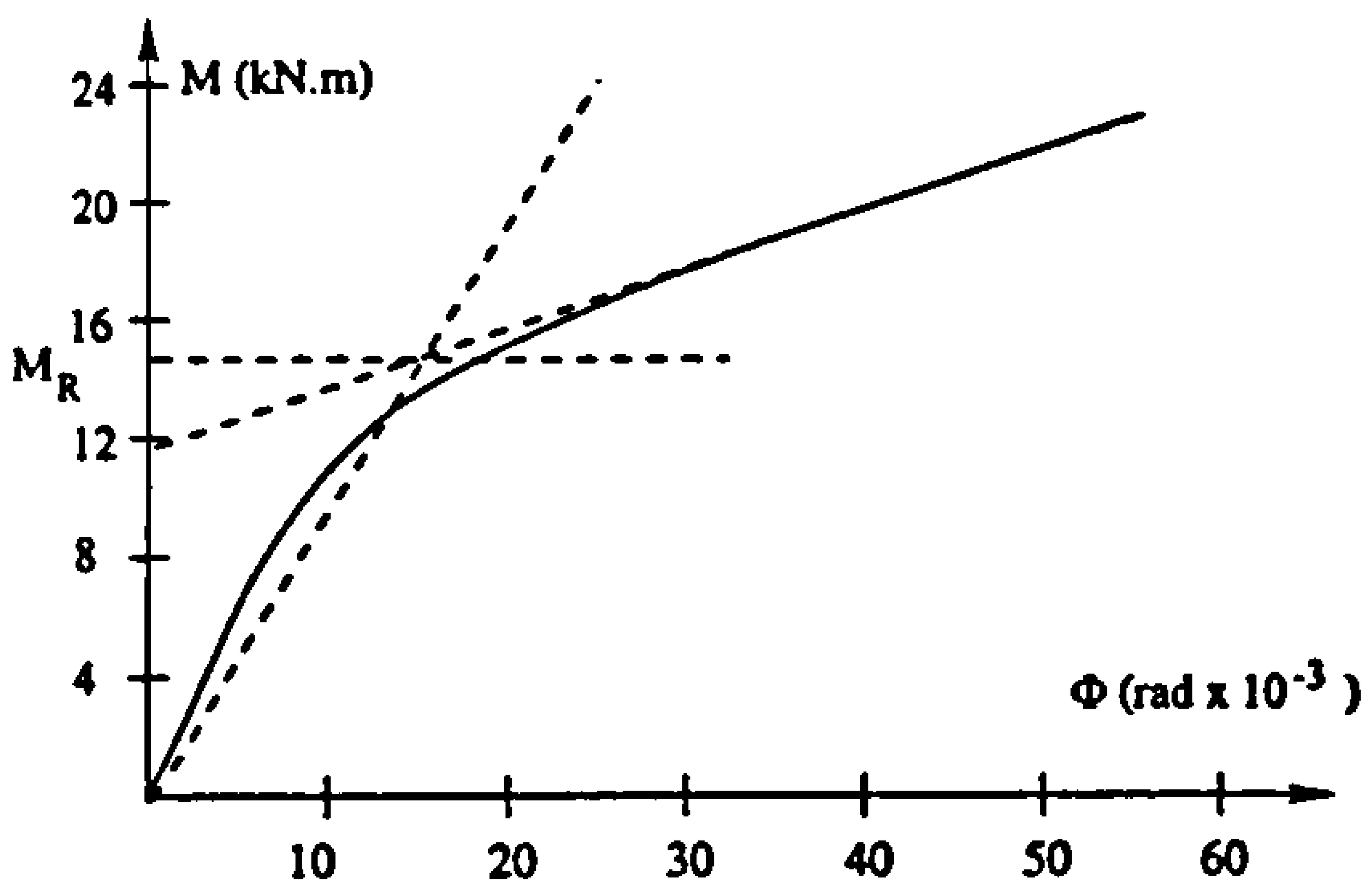

Fig. 4.2(g). Test No. 8 (M_R = 15.0 kN.m)

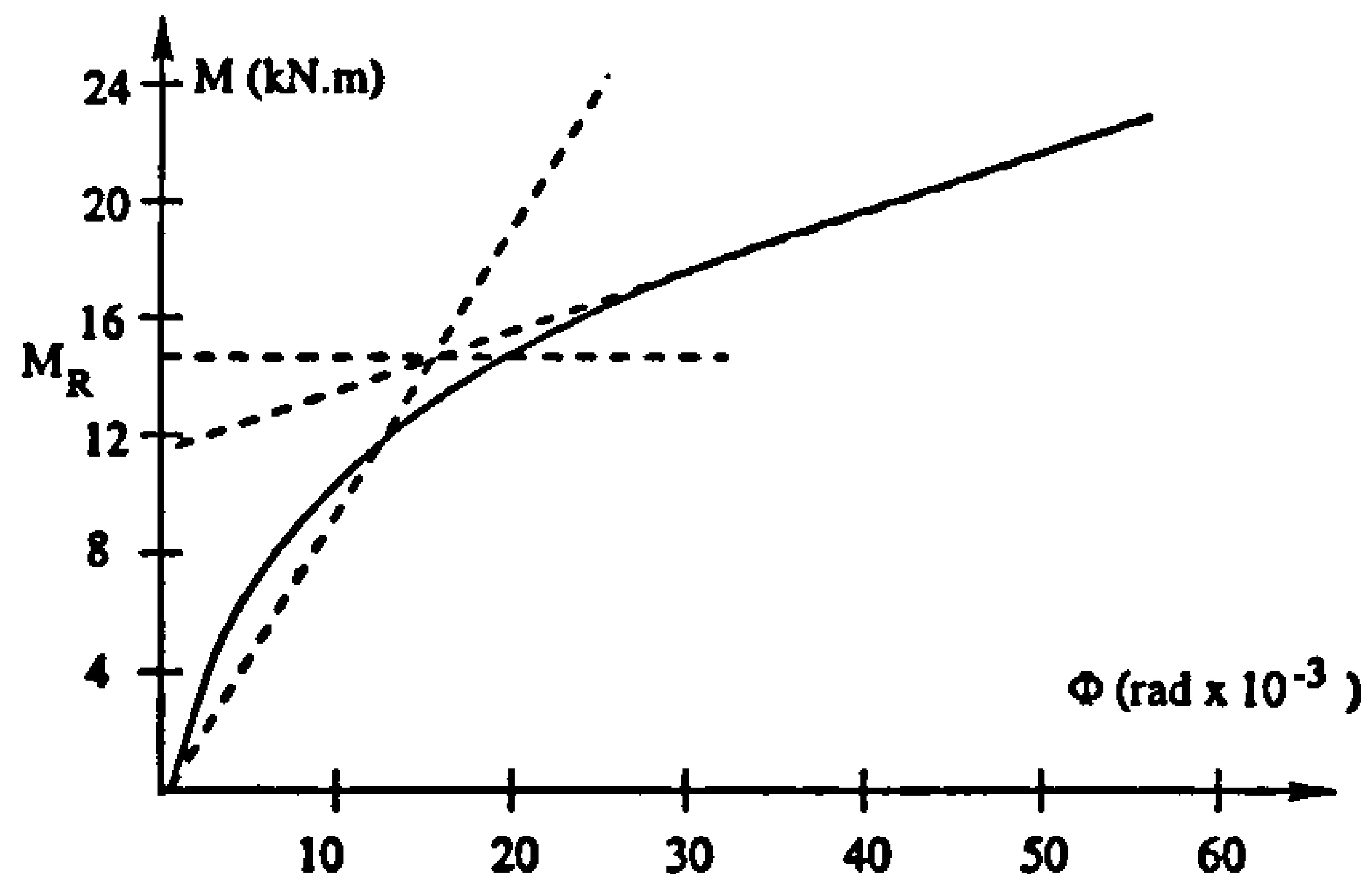

Fig. 4.2(h). Test No. 9 (M_R = 15.5 kN.m)

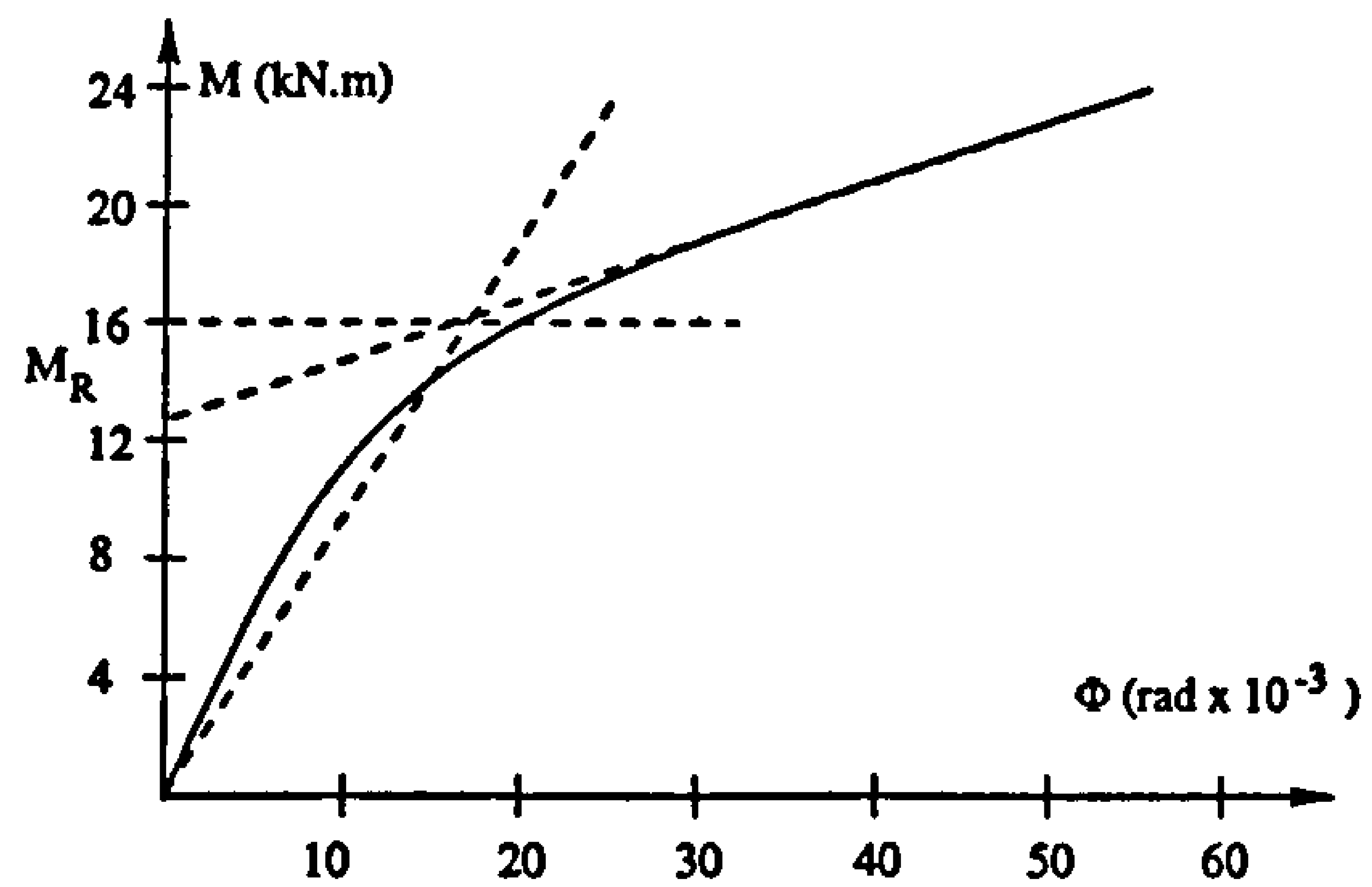

Fig. 4.2(i). Test No. 10 (M_R = 16.0 kN.m)

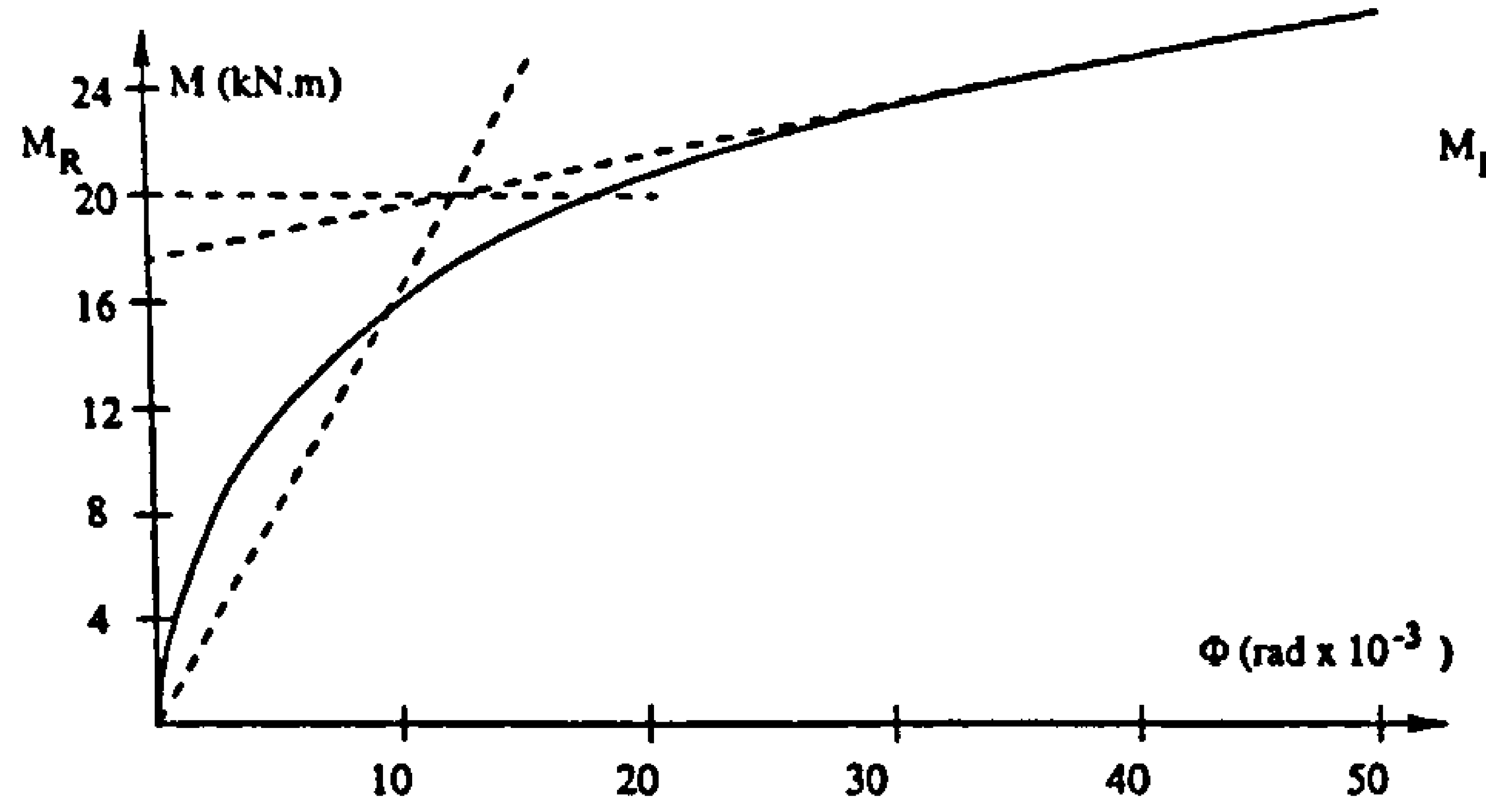

Fig. 4.2(j). Test No. 11 (M_R = 20.0 kN.m)

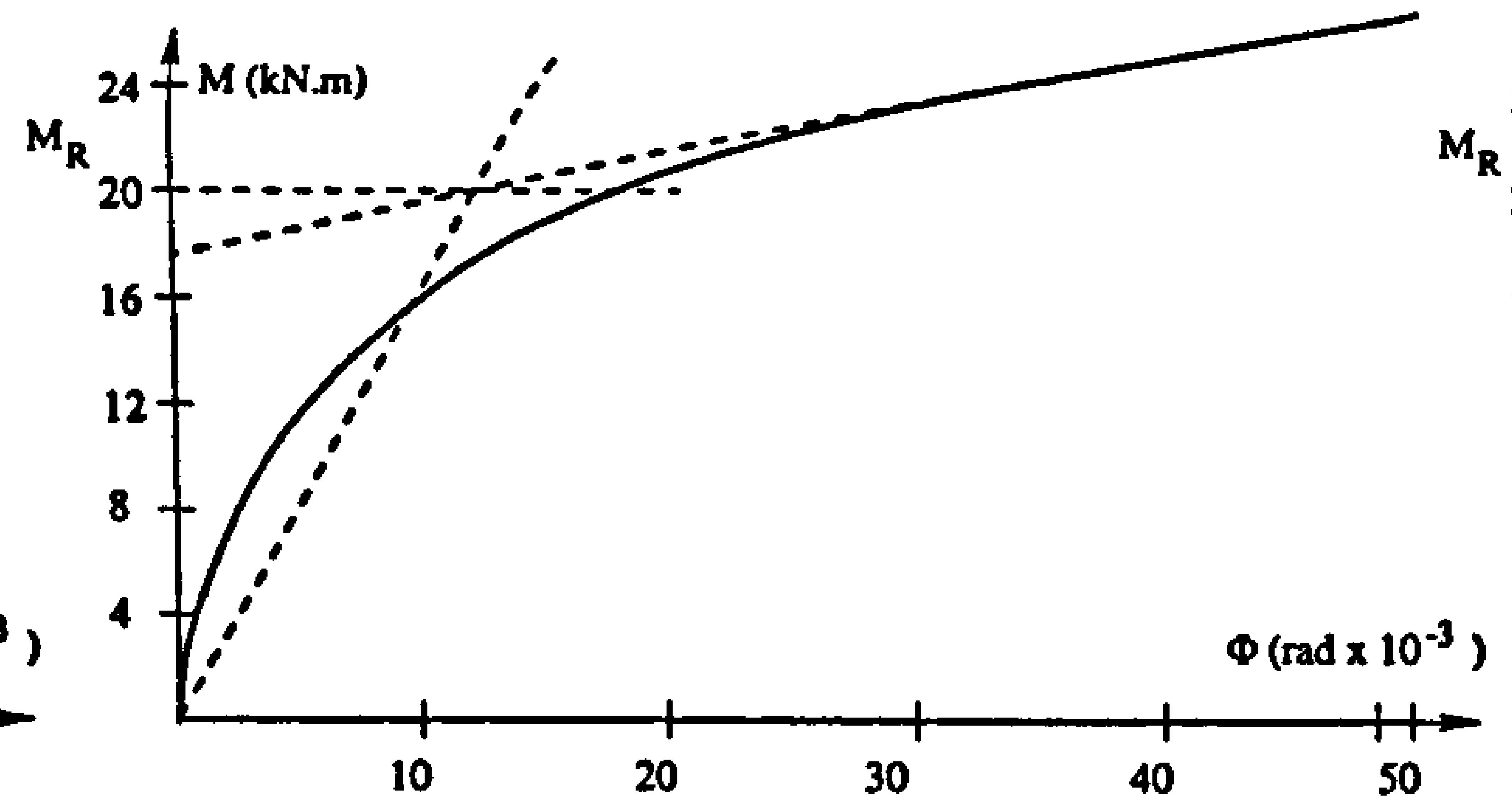

Fig. 4.2(k). Test No. 12 (M_R = 20.0 kN.m)

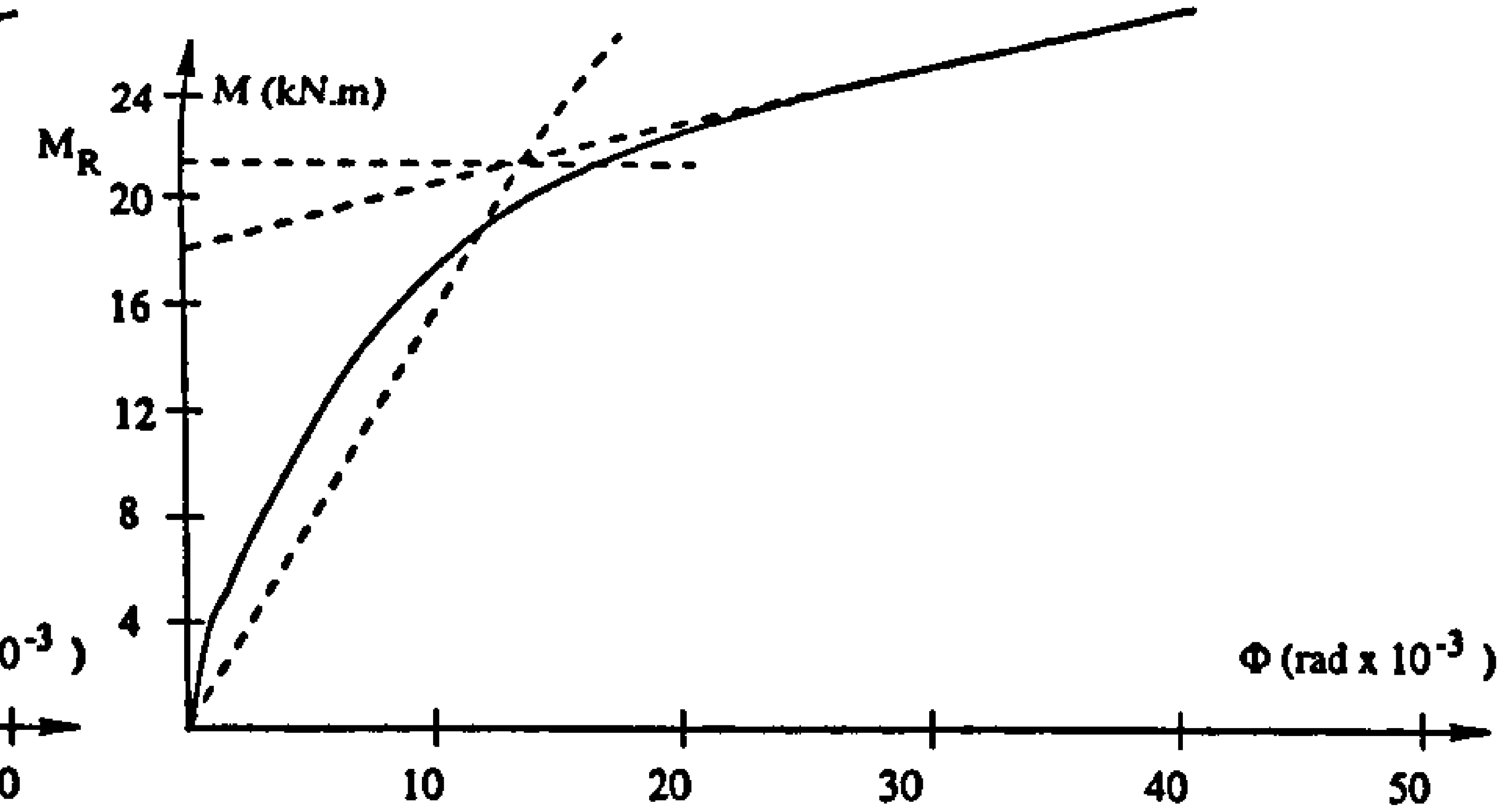

Fig. 4.2(l). Test No. 13 (M_R = 21.5 kN.m)

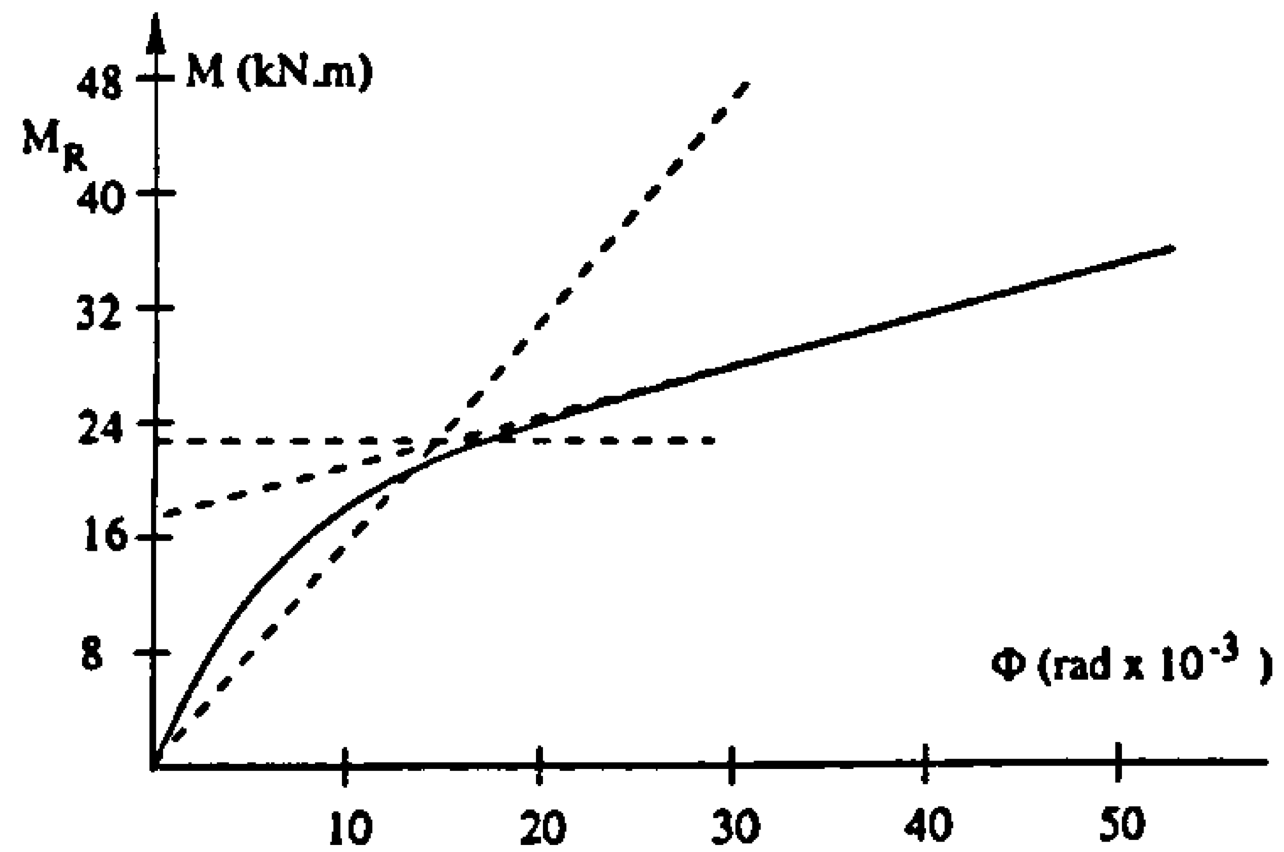

Fig. 4.2(m). Test No. 14 (M_R = 23.0 kN.m)

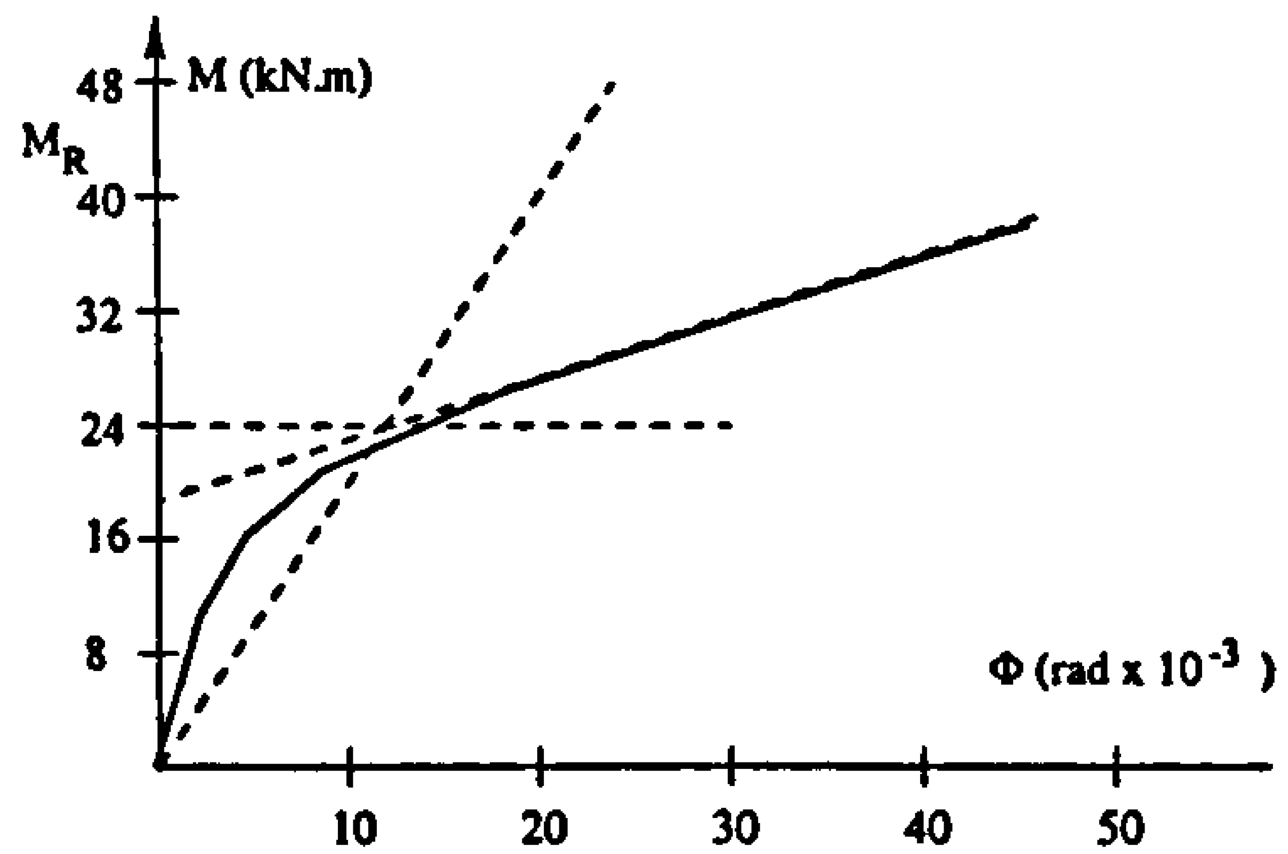

Fig. 4.2(n). Test No. 15 (M_R= 24.0 kN.m)

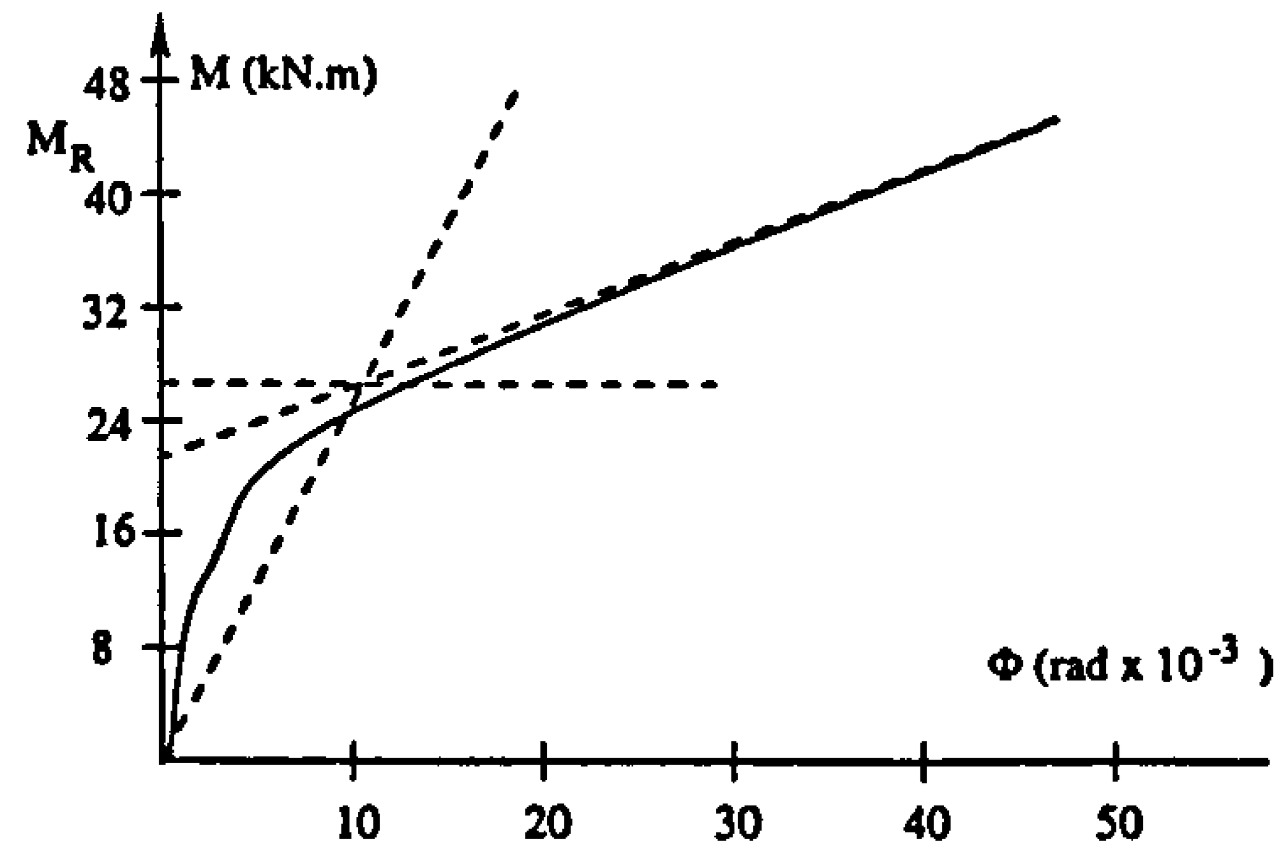

Fig. 4.2(o). Test No. 16 (M_R = 27.5 kN.m)

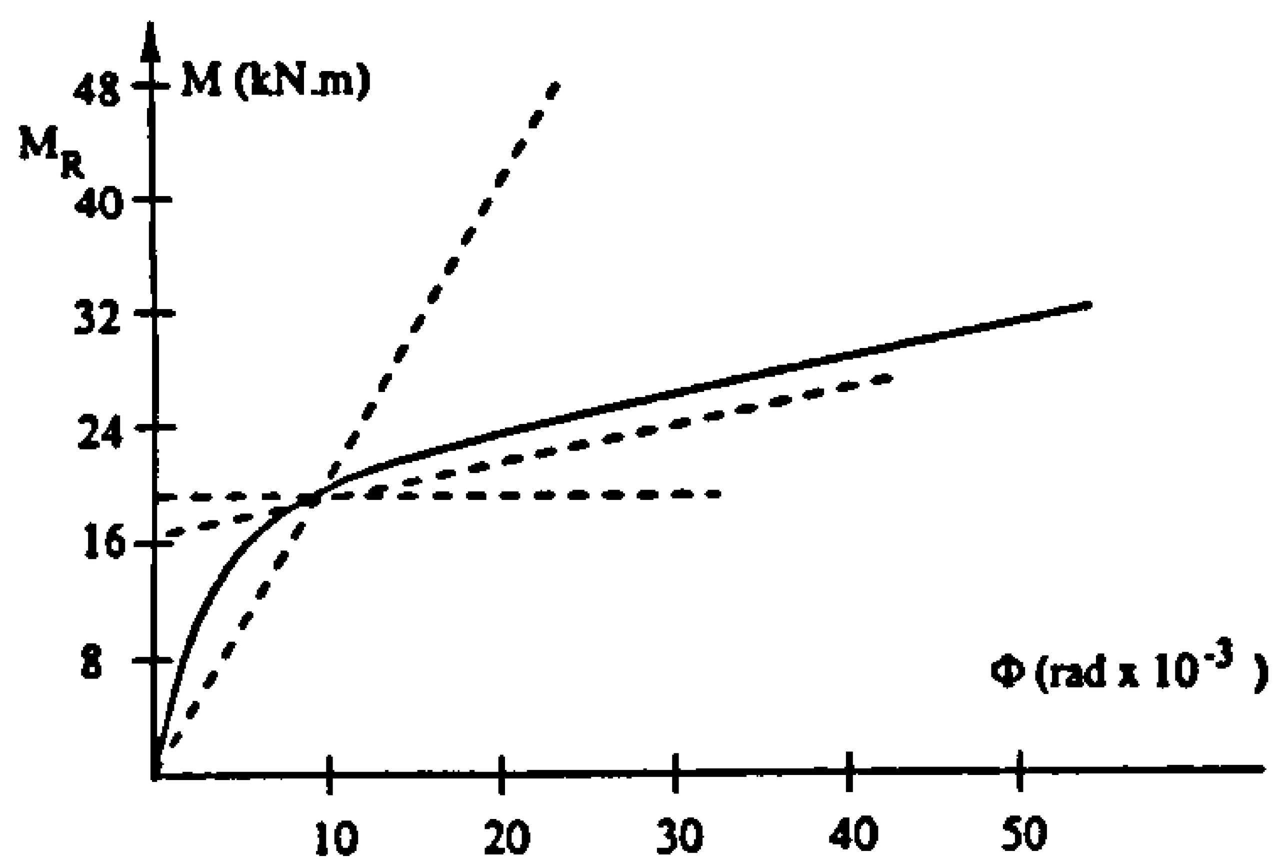

Fig. 4.2(p). Test No. 17 (M_R = 19.0 kN.m)

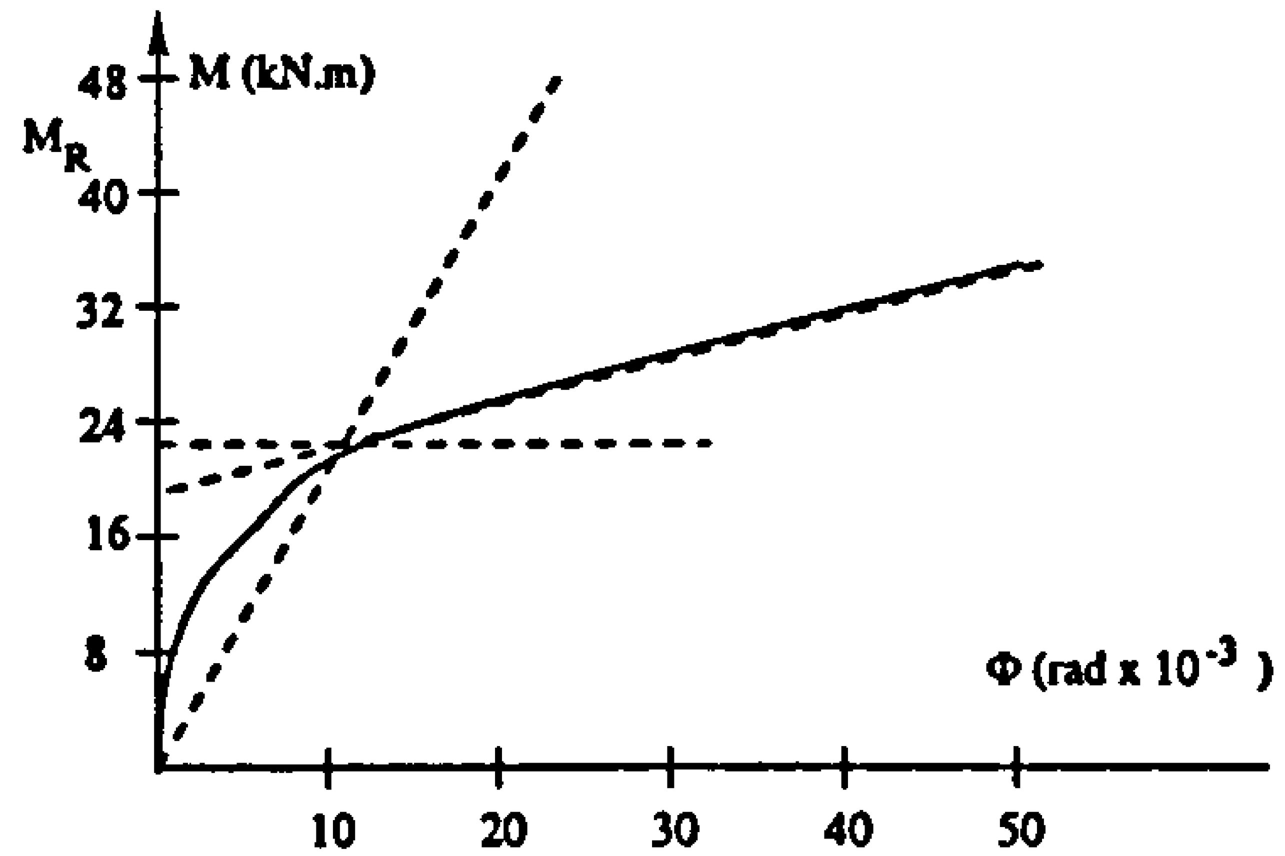

Fig. 4.2(q). Test No. 18 (M_R = 23.0 kN.m)

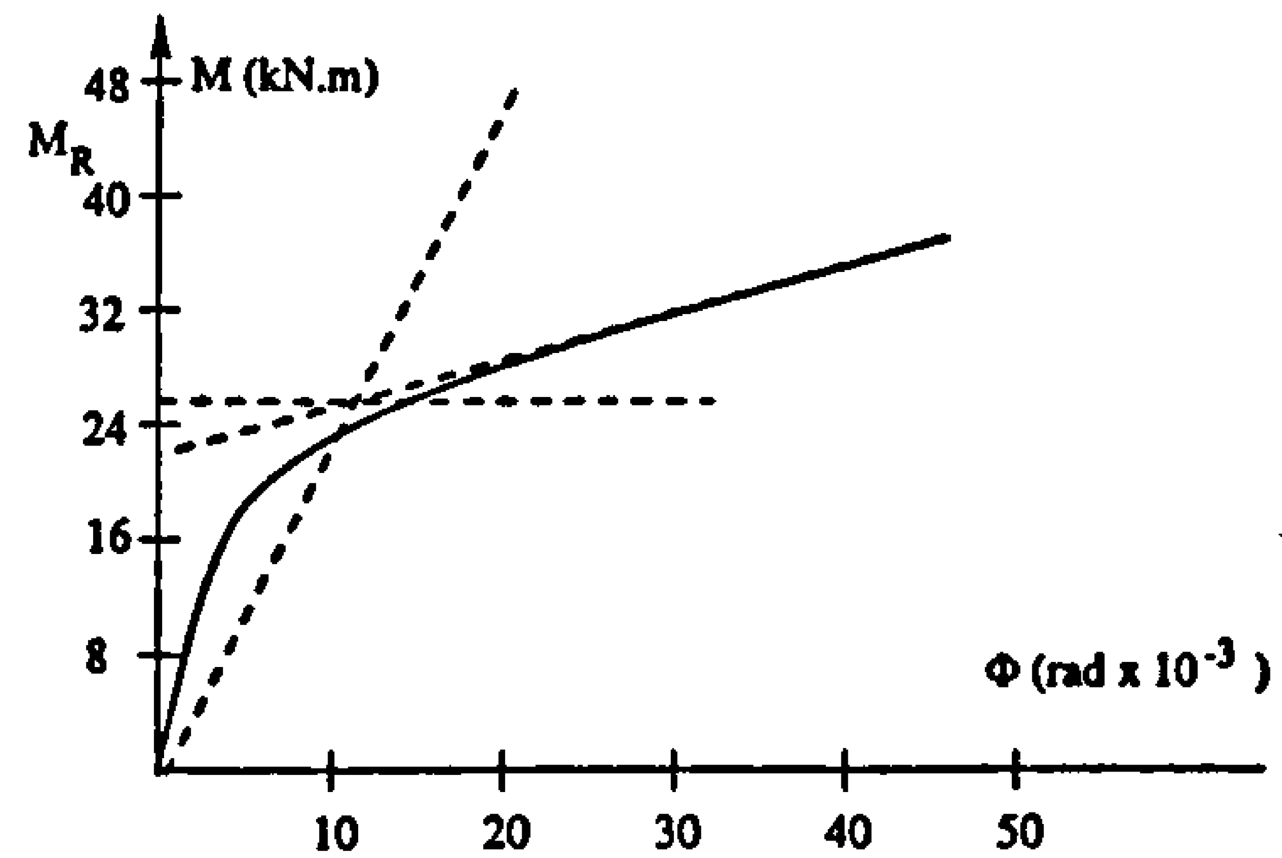

Fig. 4.2(r). Test No. 19 (M_R = 25.0 kN.m)

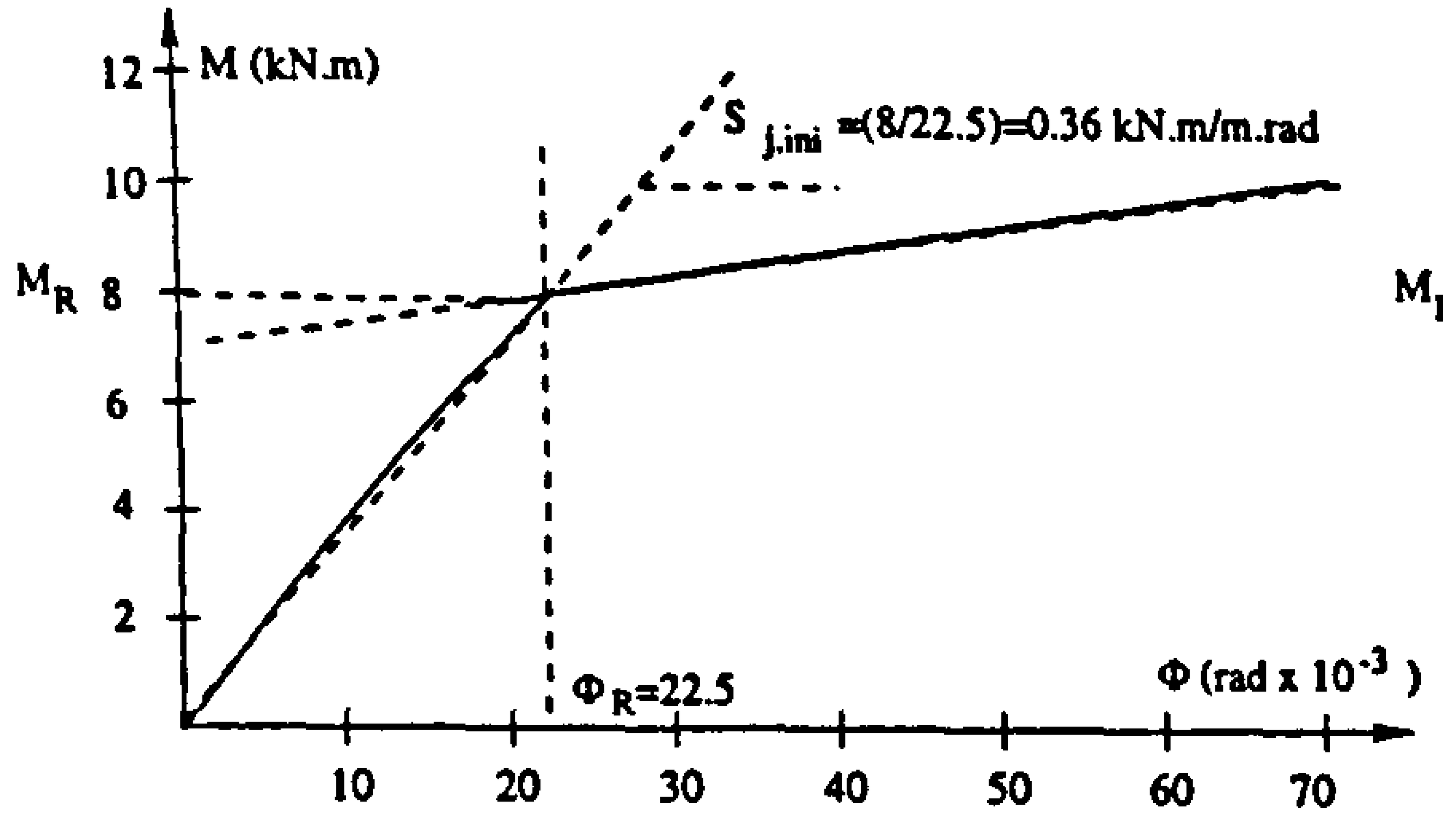

Fig. 4.3(a). Test No. 2 ($S_{j,ini}$ = 0.36 kN.m/m.rad)

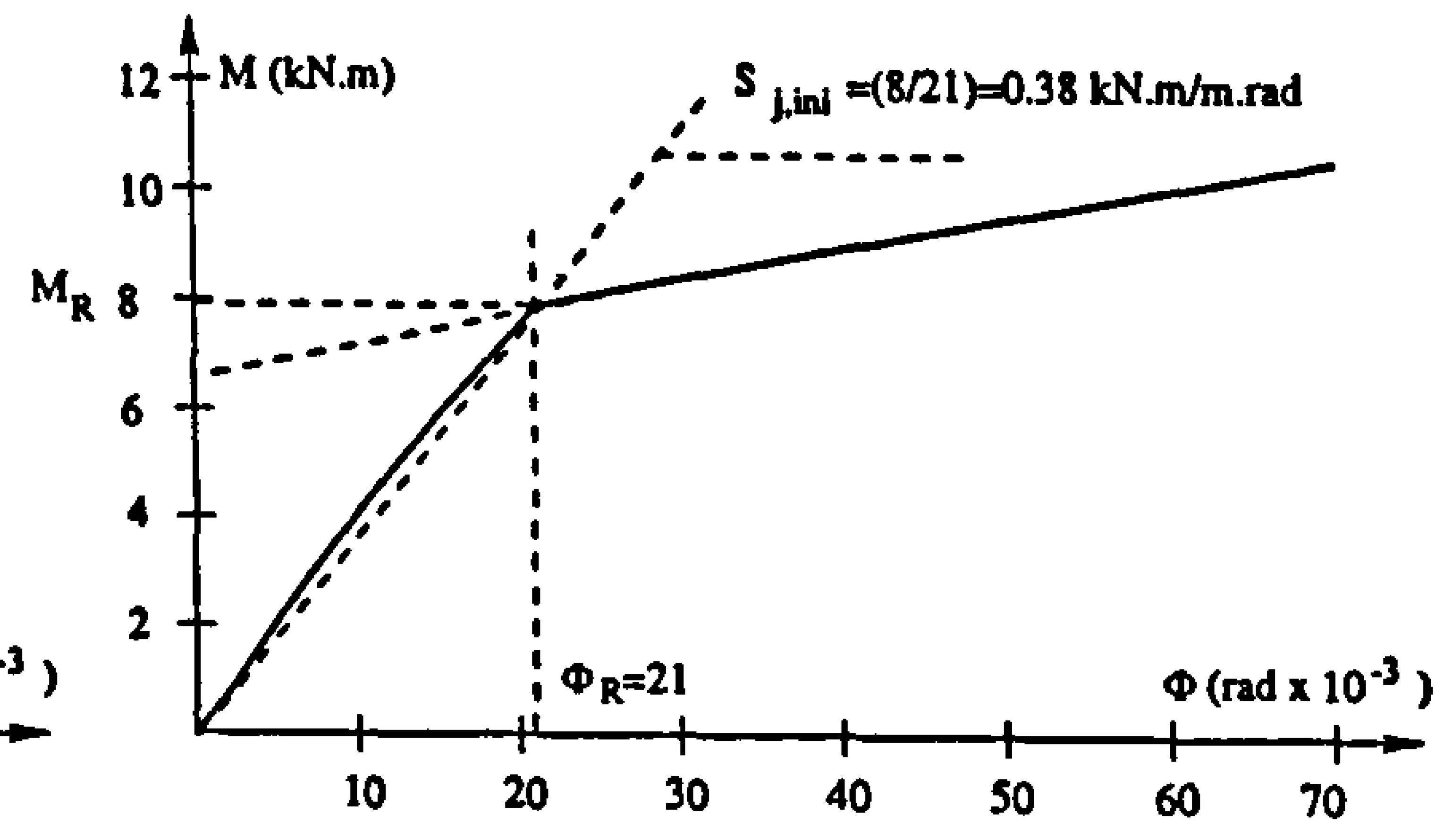

Fig. 4.3(b). Test No. 3 ($S_{j,ini}$ = 0.38 kN.m/m.rad)

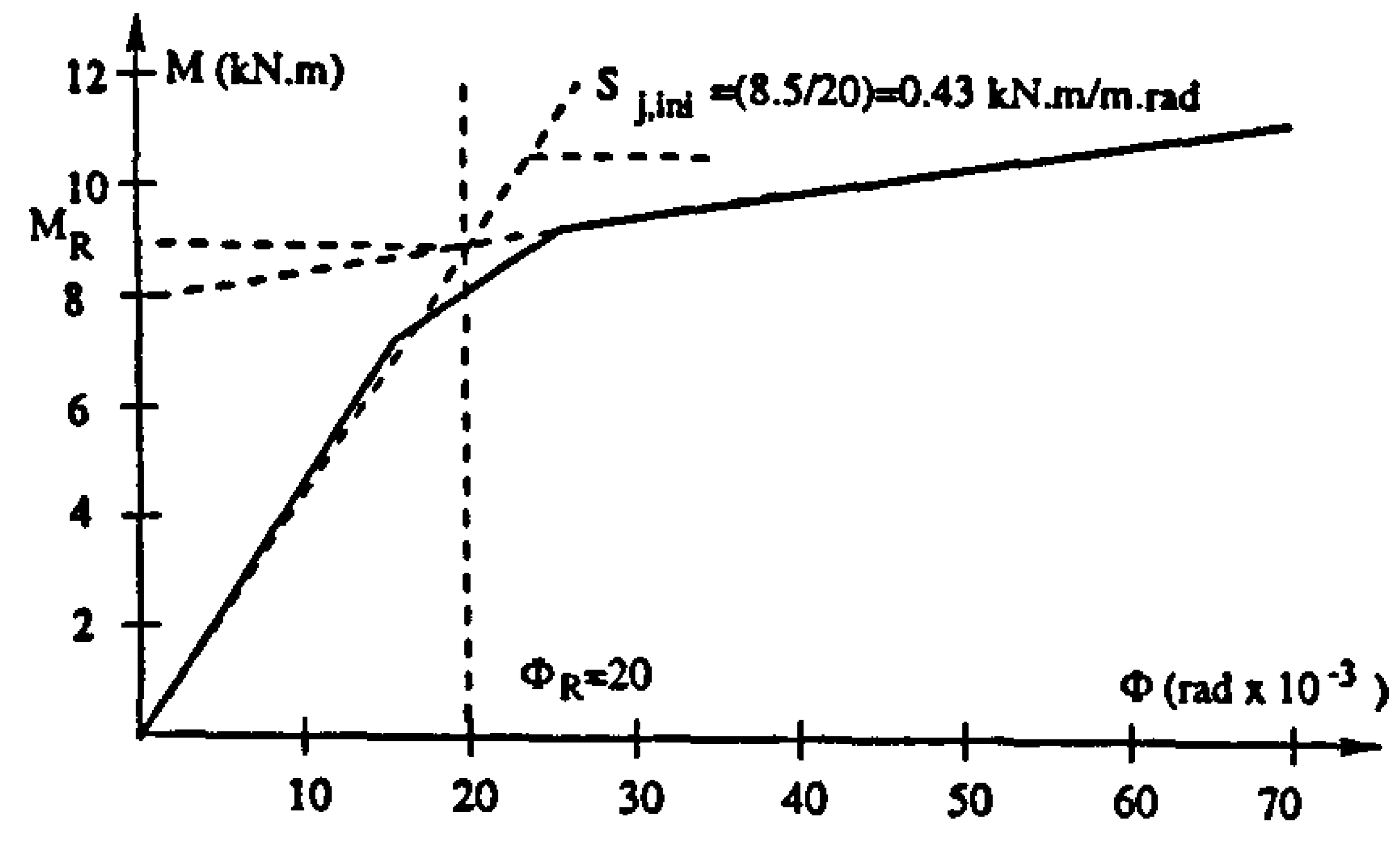

Fig. 4.3(c). Test No. 4 ($S_{j,ini}$ = 0.43 kN.m/m.rad)

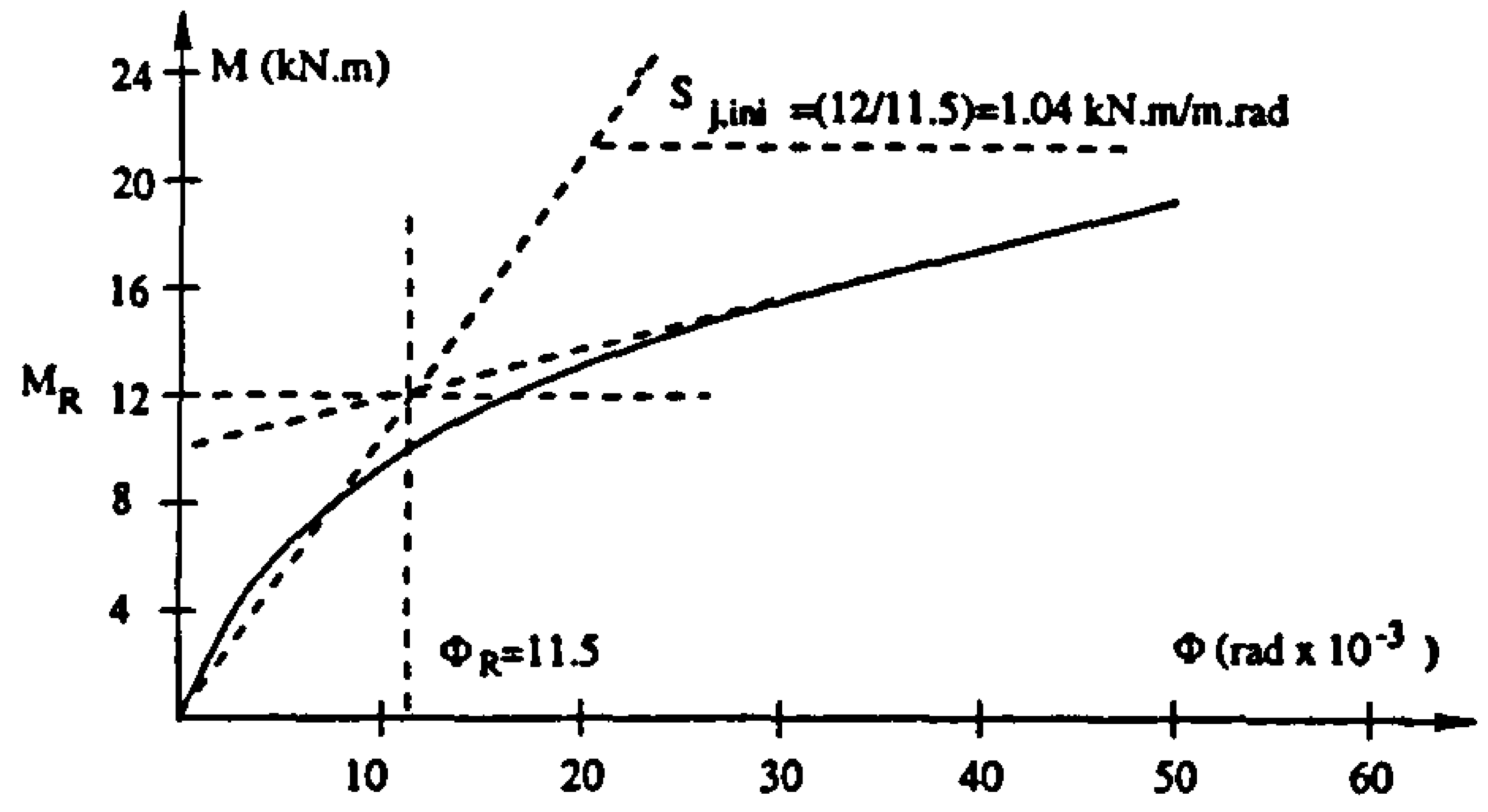

Fig. 4.3(d). Test No. 5 ($S_{j,ini}$ = 1.04 kN.m/m.rad)

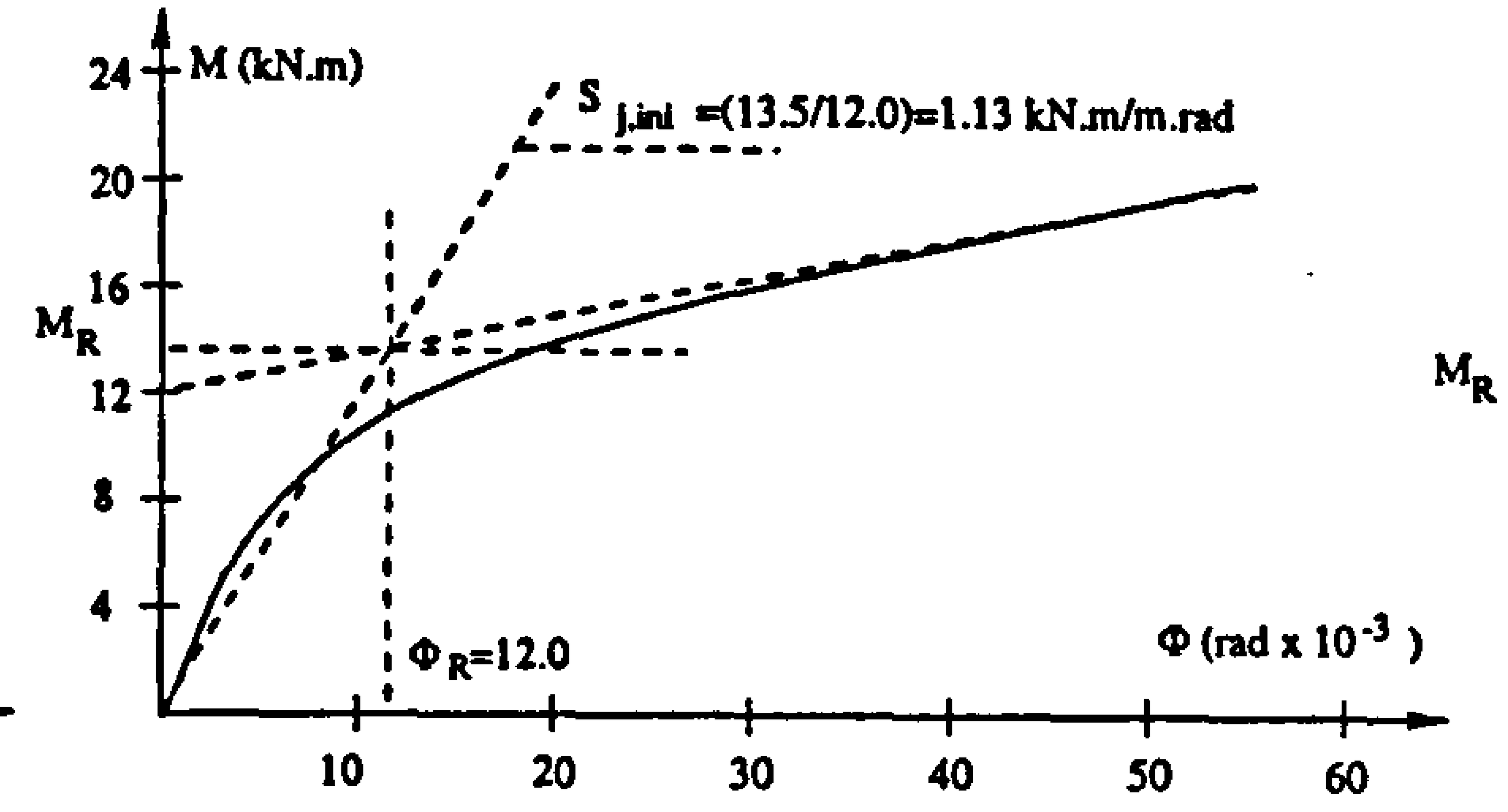

Fig. 4.3(e). Test No. 6 ($S_{j,ini}$ = 1.13 kN.m/m.rad)

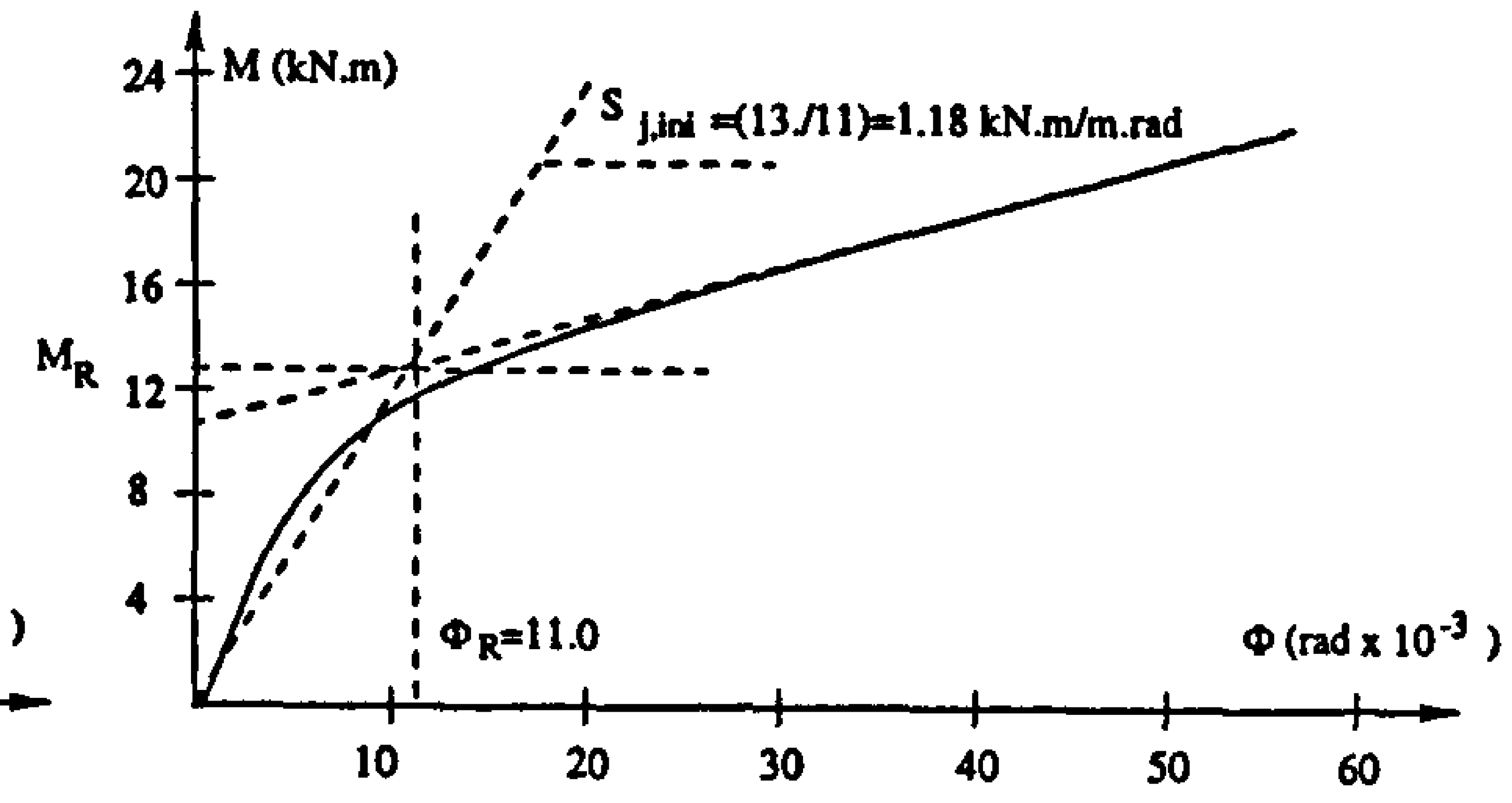

Fig. 4.3(f). Test No. 7 ($S_{j,ini}$ = 1.18 kN.m/m.rad)

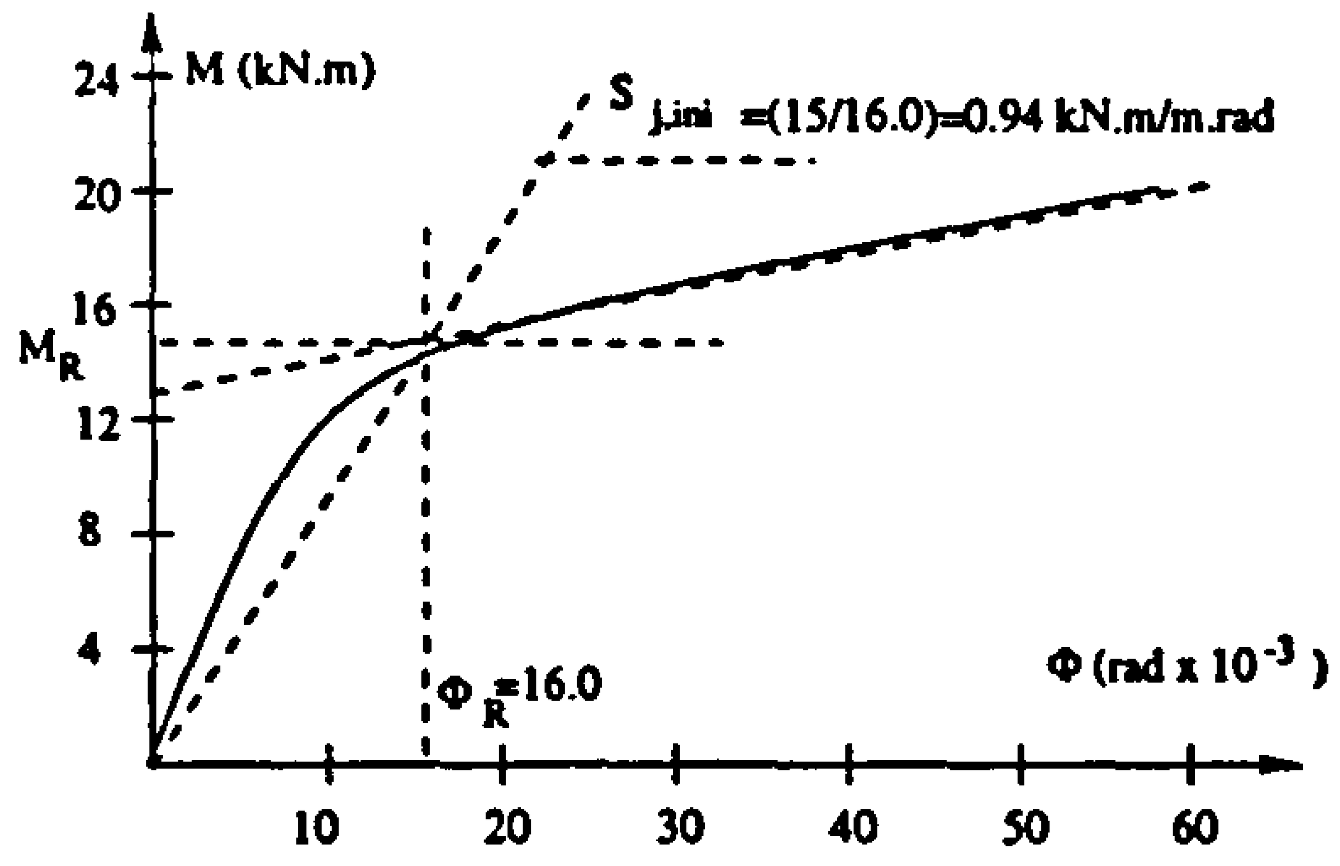

Fig. 4.3(g). Test No. 8 ($S_{j,ini}$ = 0.94 kN.m/m.rad)

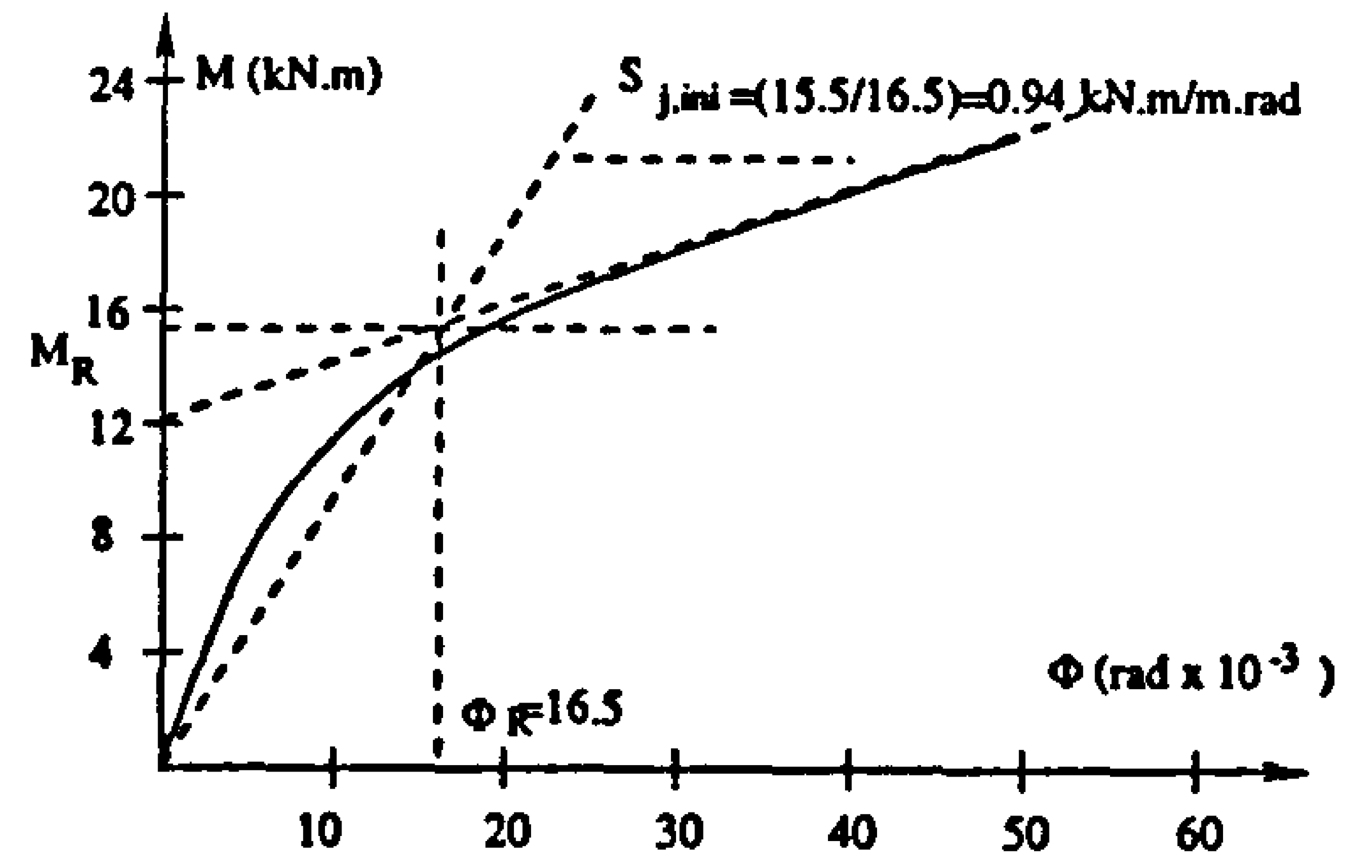

Fig. 4.3(h). Test No. 9 ($S_{j,ini}$ = 0.94 kN.m/m.rad)

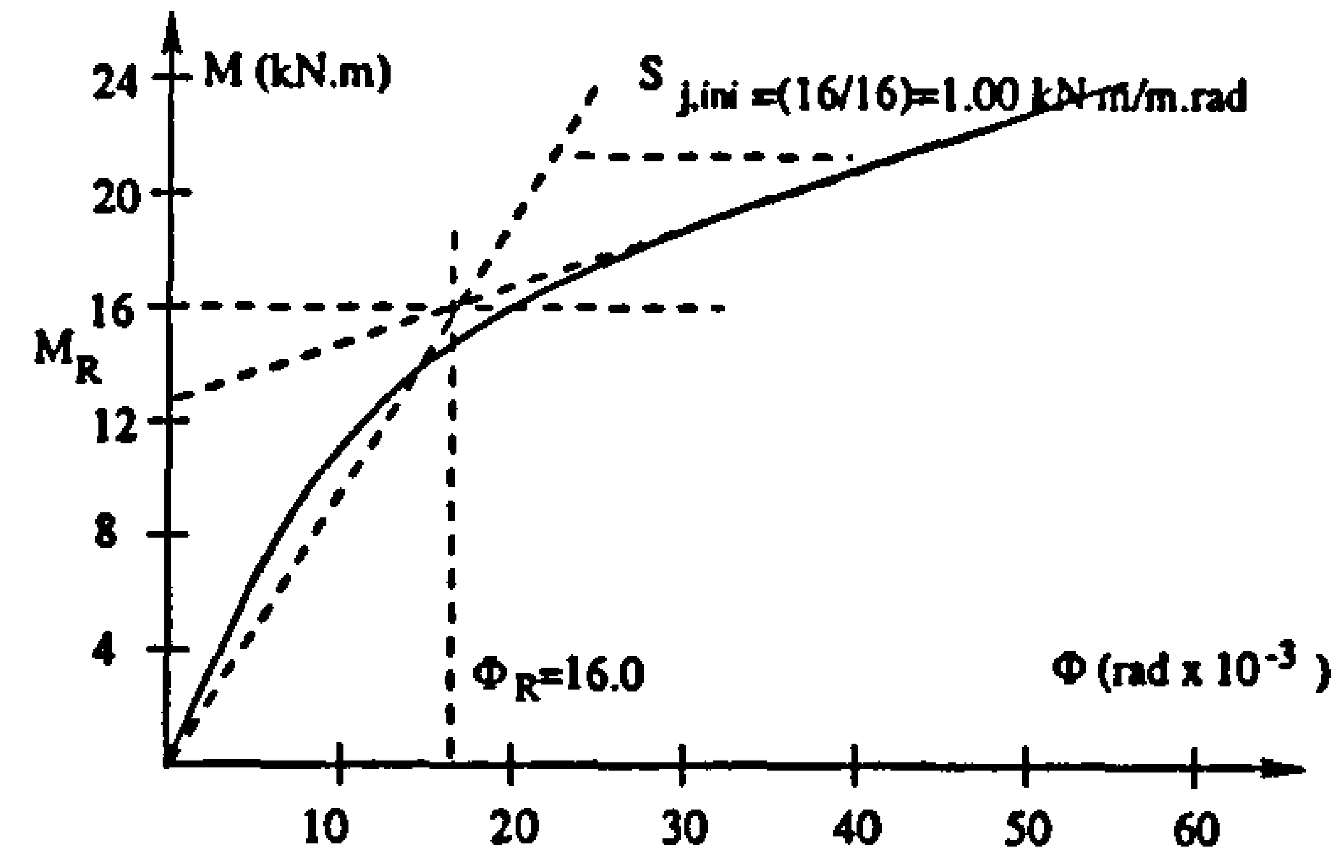

Fig. 4.3(i). Test No. 10 ($S_{j,ini}$ = 1.00 kN.m/m.rad)

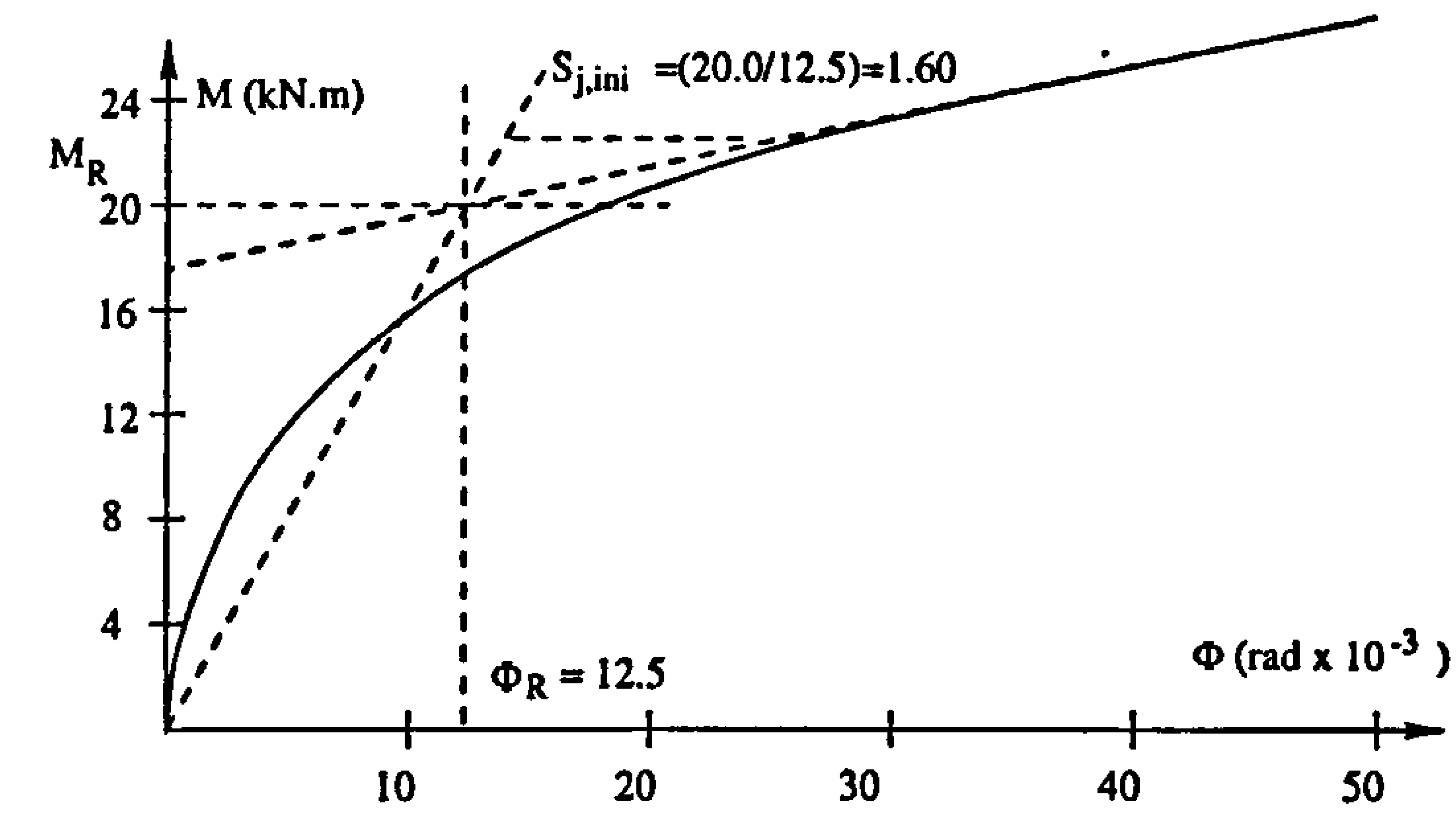

Fig. 4.3(j). Test No. 11 ($S_{j,ini}$ = 1.60 kN.m/m.rad)

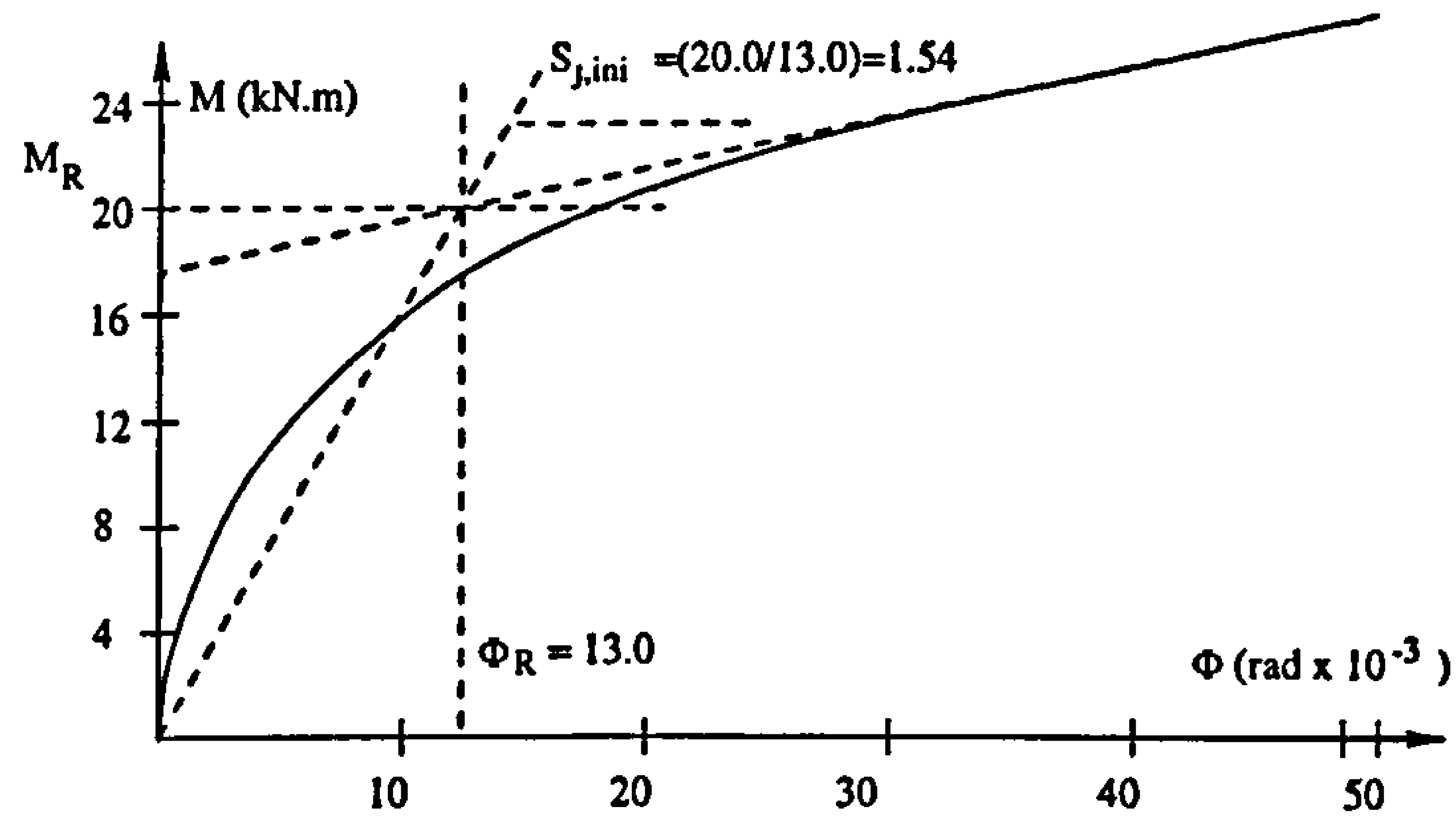

Fig. 4.3(k). Test No. 12 ($S_{j,ini}$ = 1.54 kN.m/m.rad)

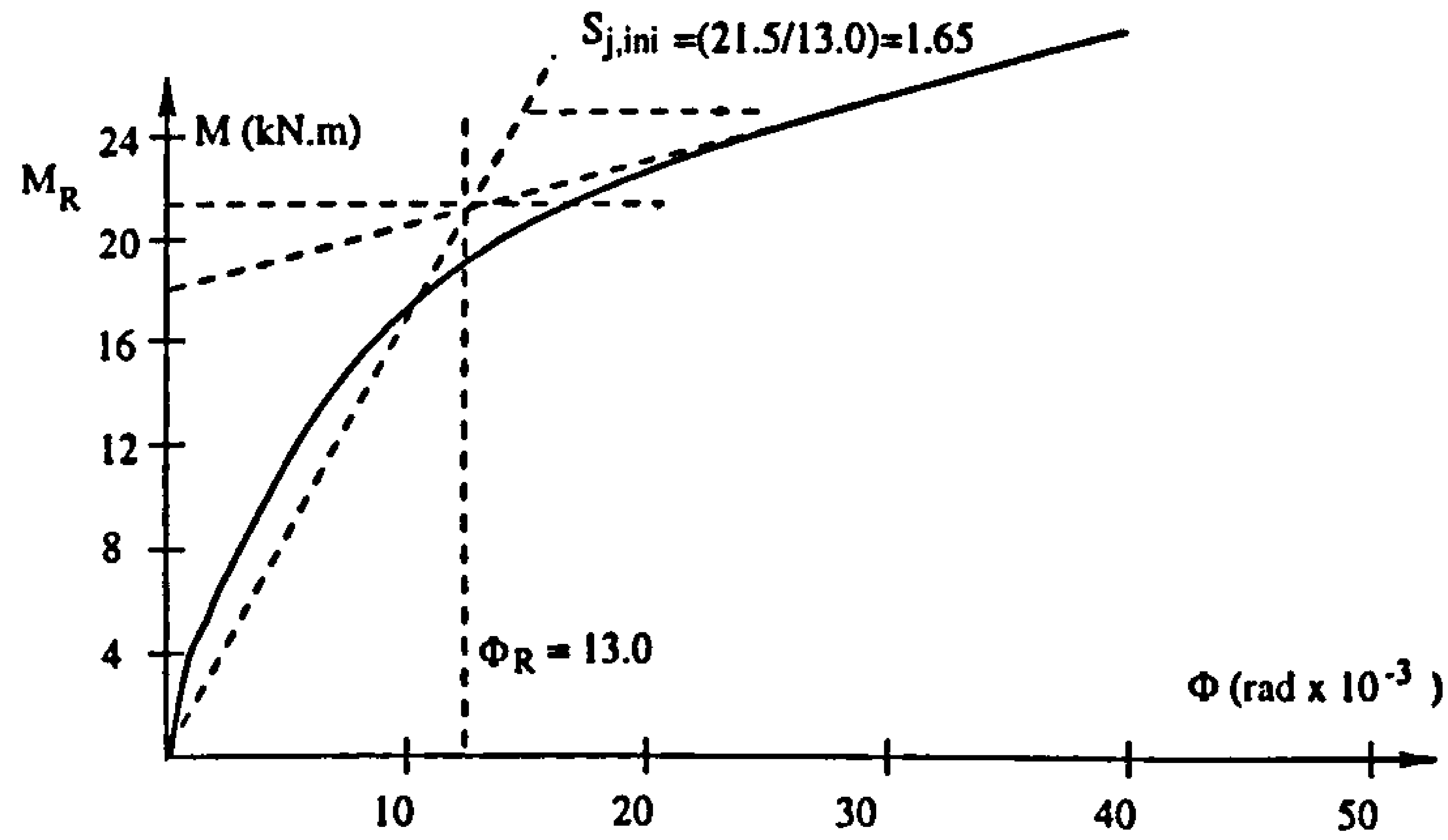

Fig. 4.3(l). Test No. 13 ($S_{j,ini}$ = 1.65 kN.m/m.rad)

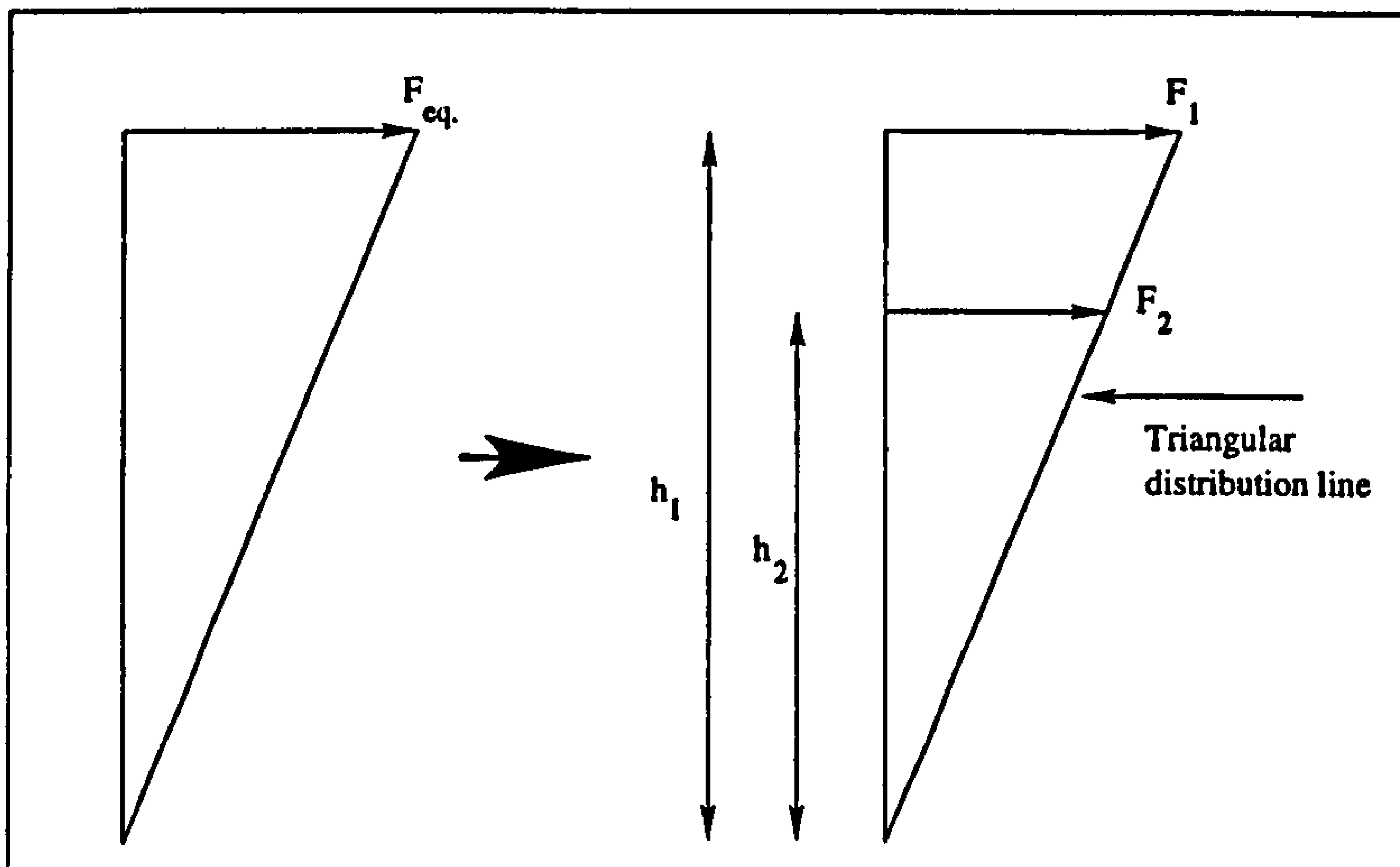

Fig 4.4 Conversion of $F_{eq.}$ to F_1 and F_2

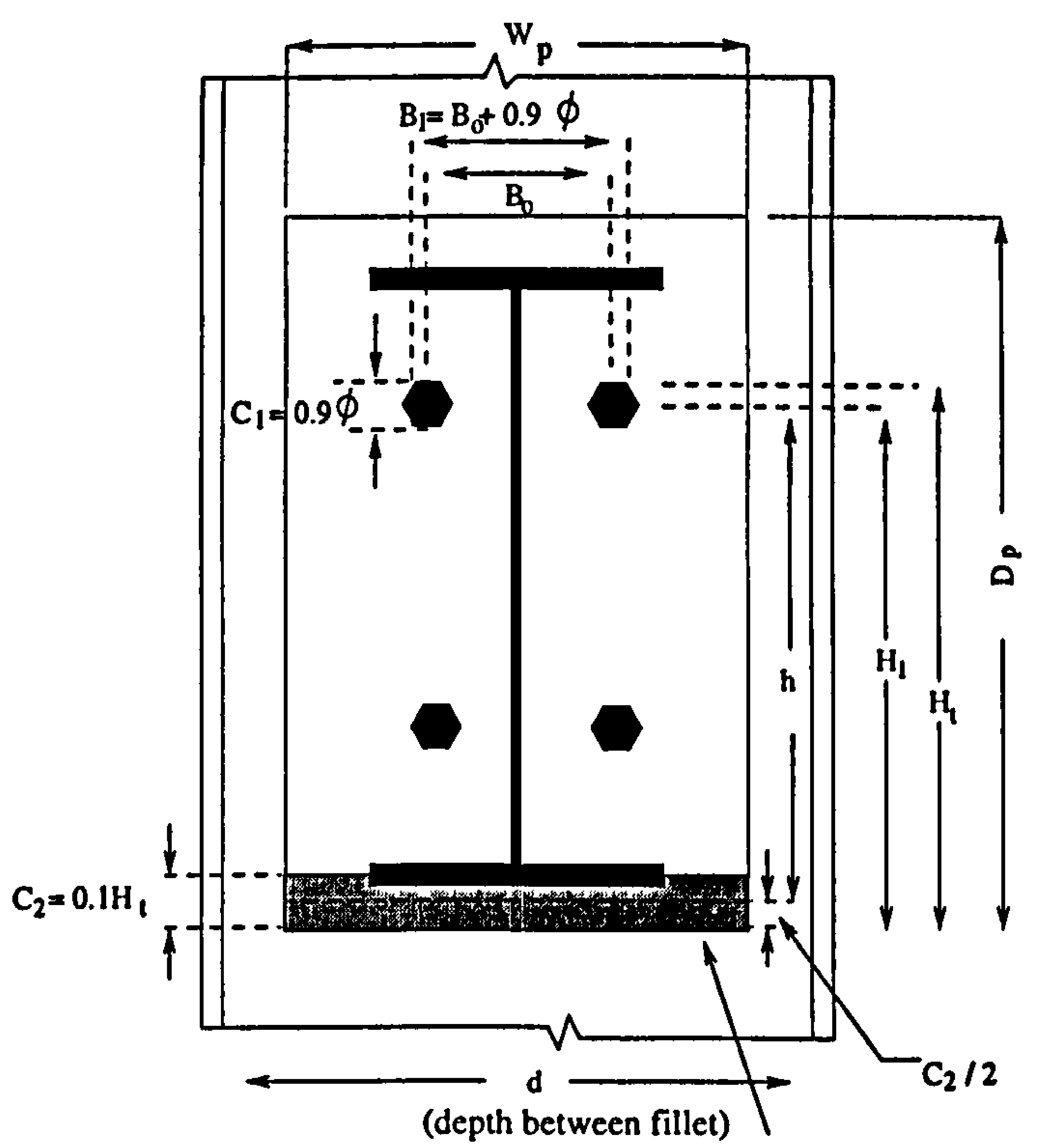

Fig 4.5 : End-plate dimensions

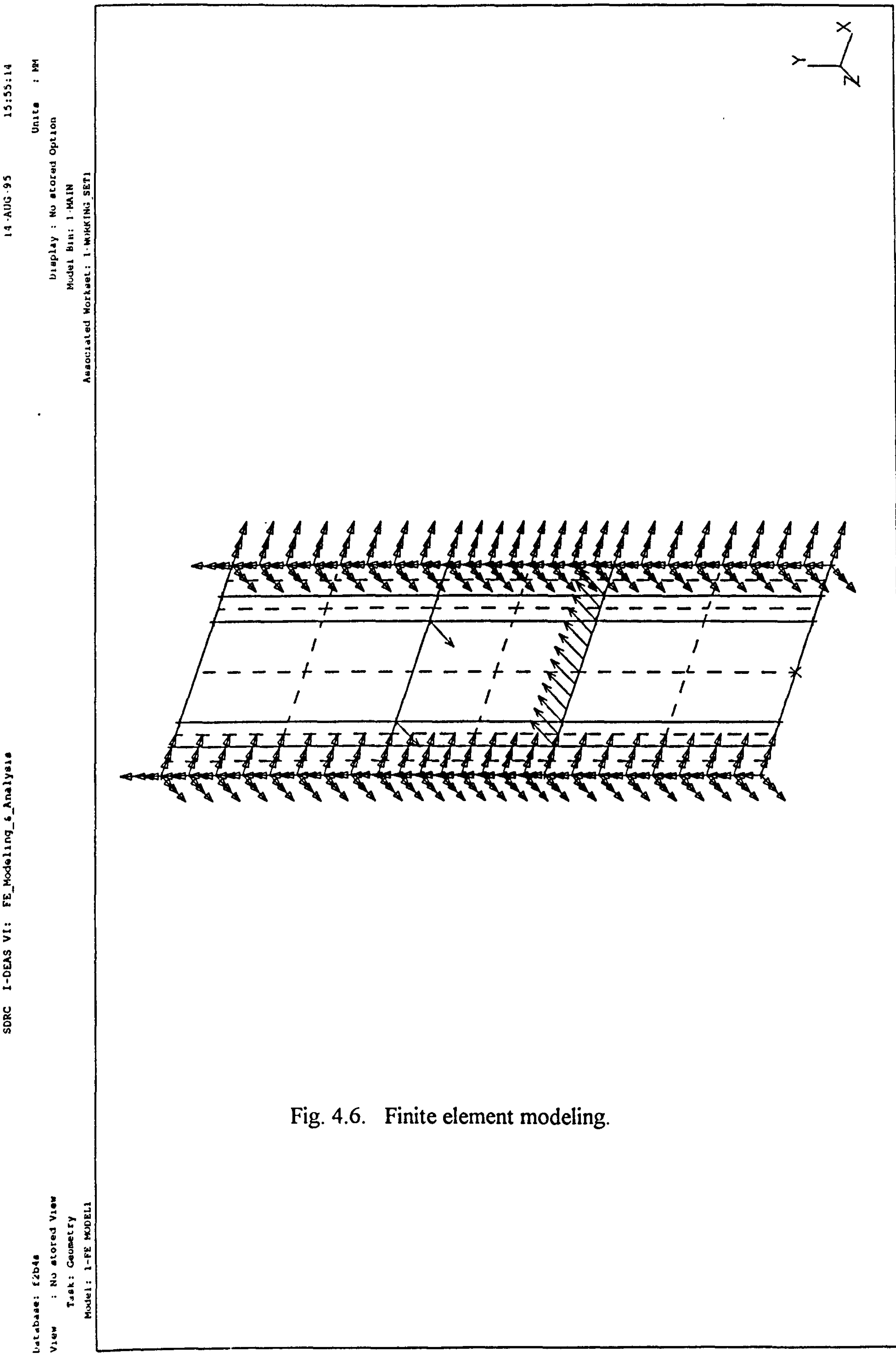

Fig. 4.6. Finite element modeling.

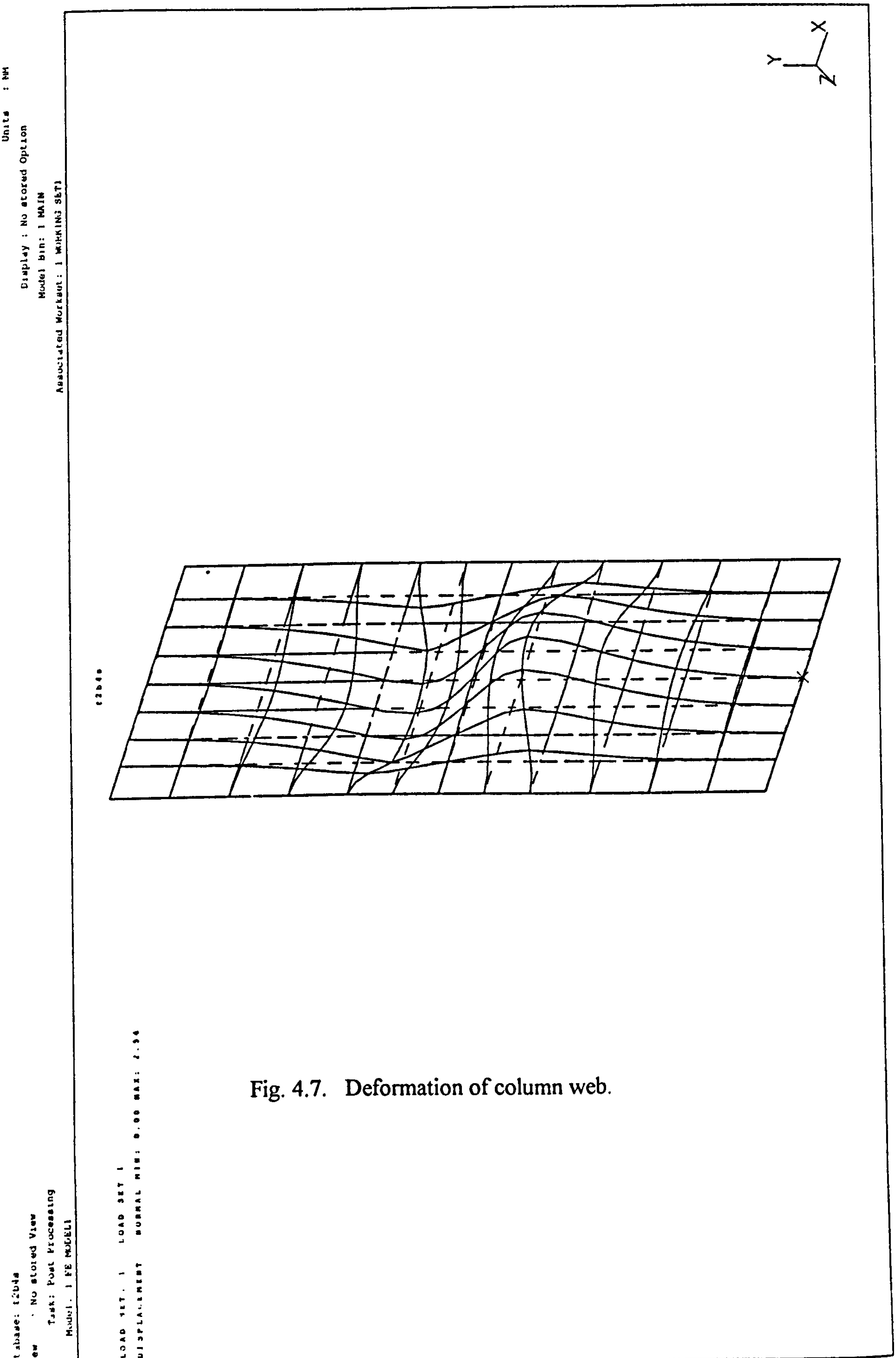

Fig. 4.7. Deformation of column web.

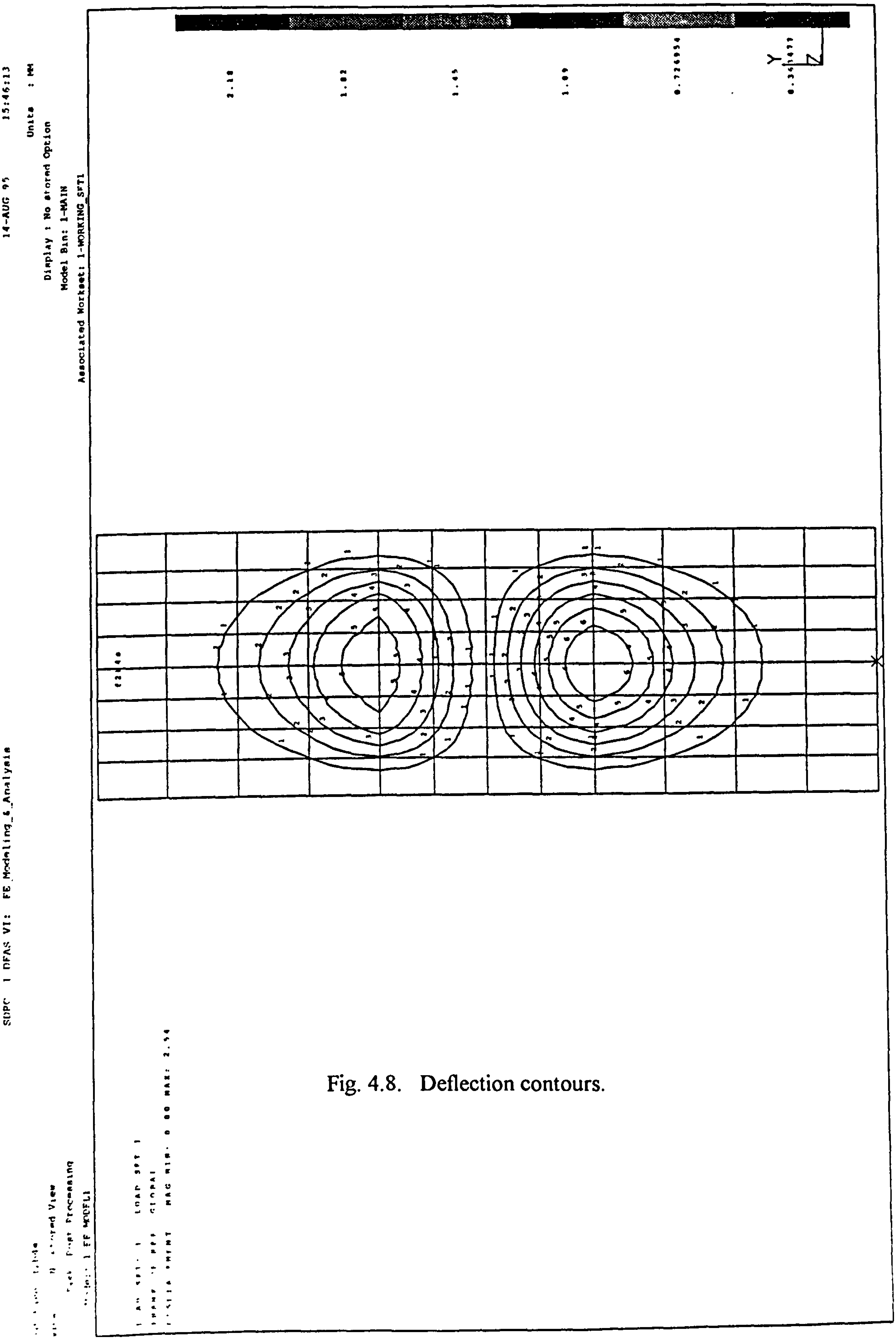

Fig. 4.8. Deflection contours.

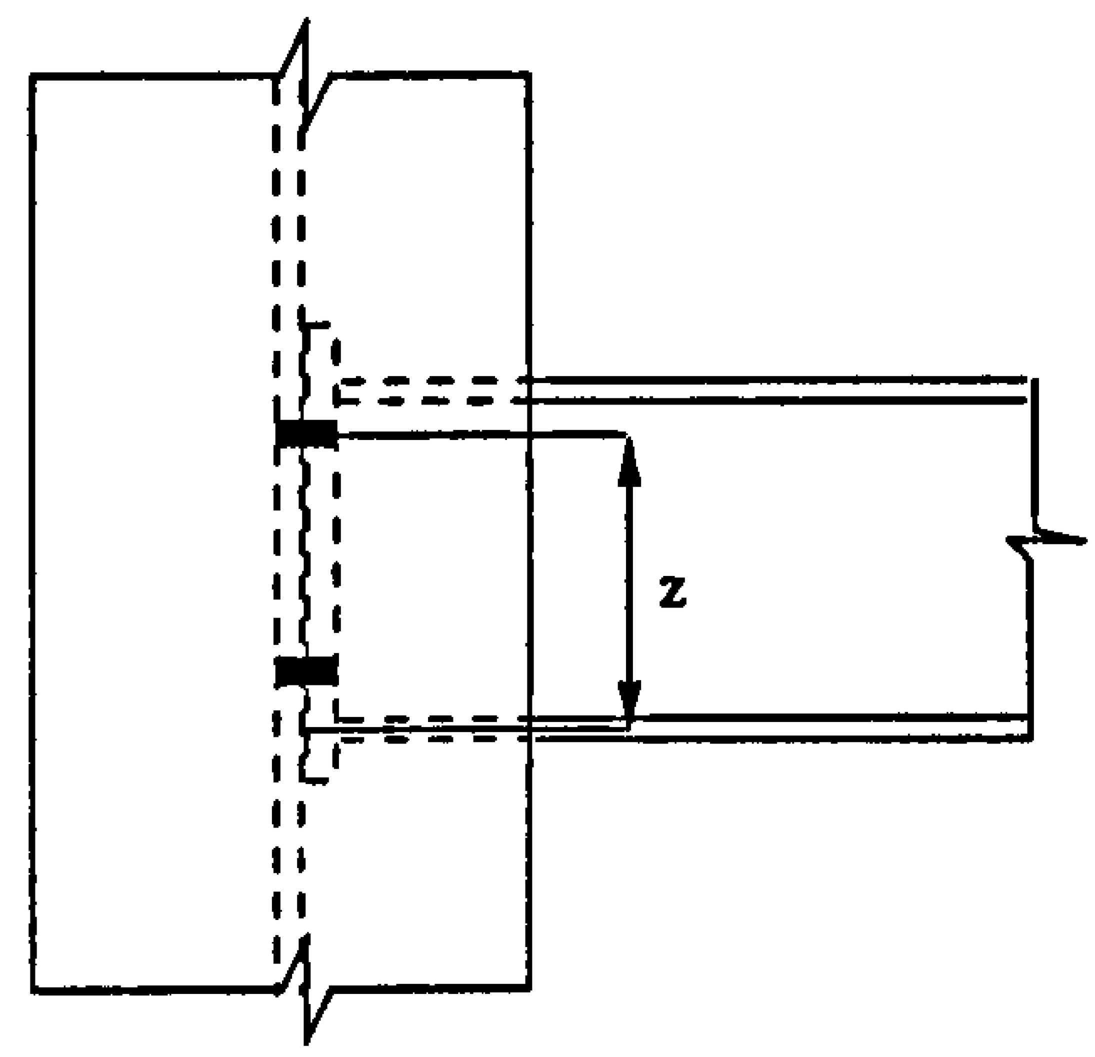

Fig. 4.9(a) Lever arm for rotation in two bolt rows.

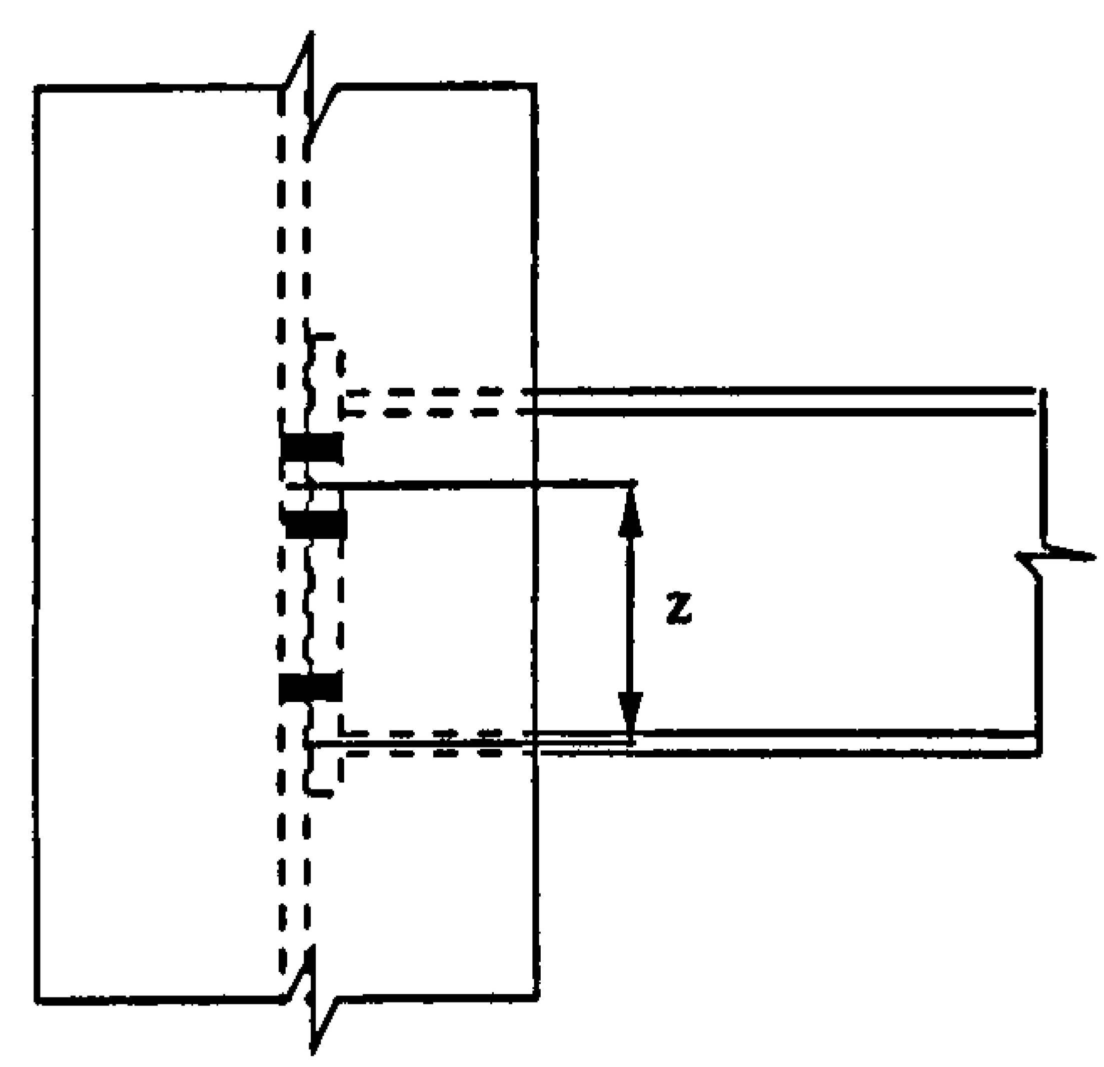

Fig. 4.9(b) Lever arm for rotation in three bolt rows.

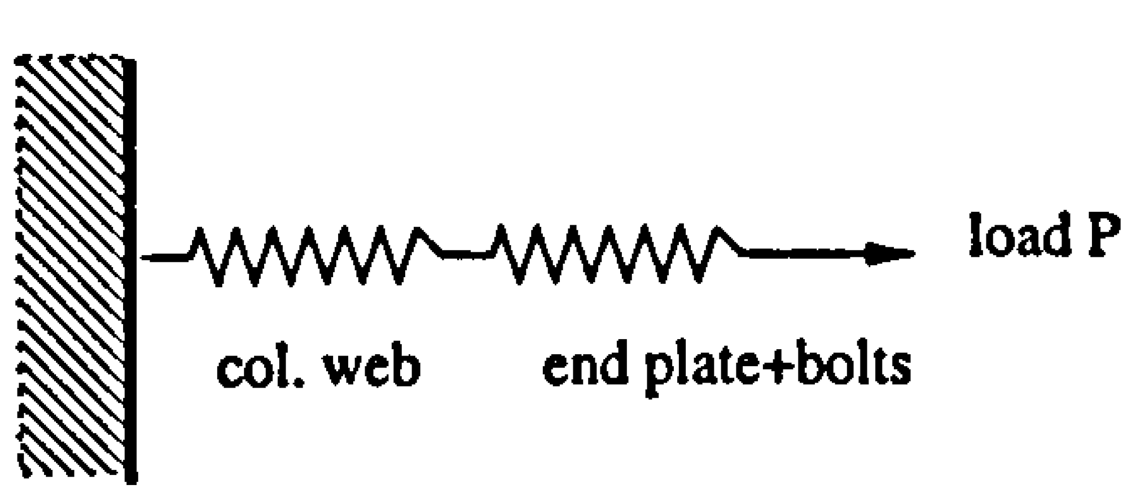

Fig. 4.10 Spring model for stiffness of column web and end plate + bolts

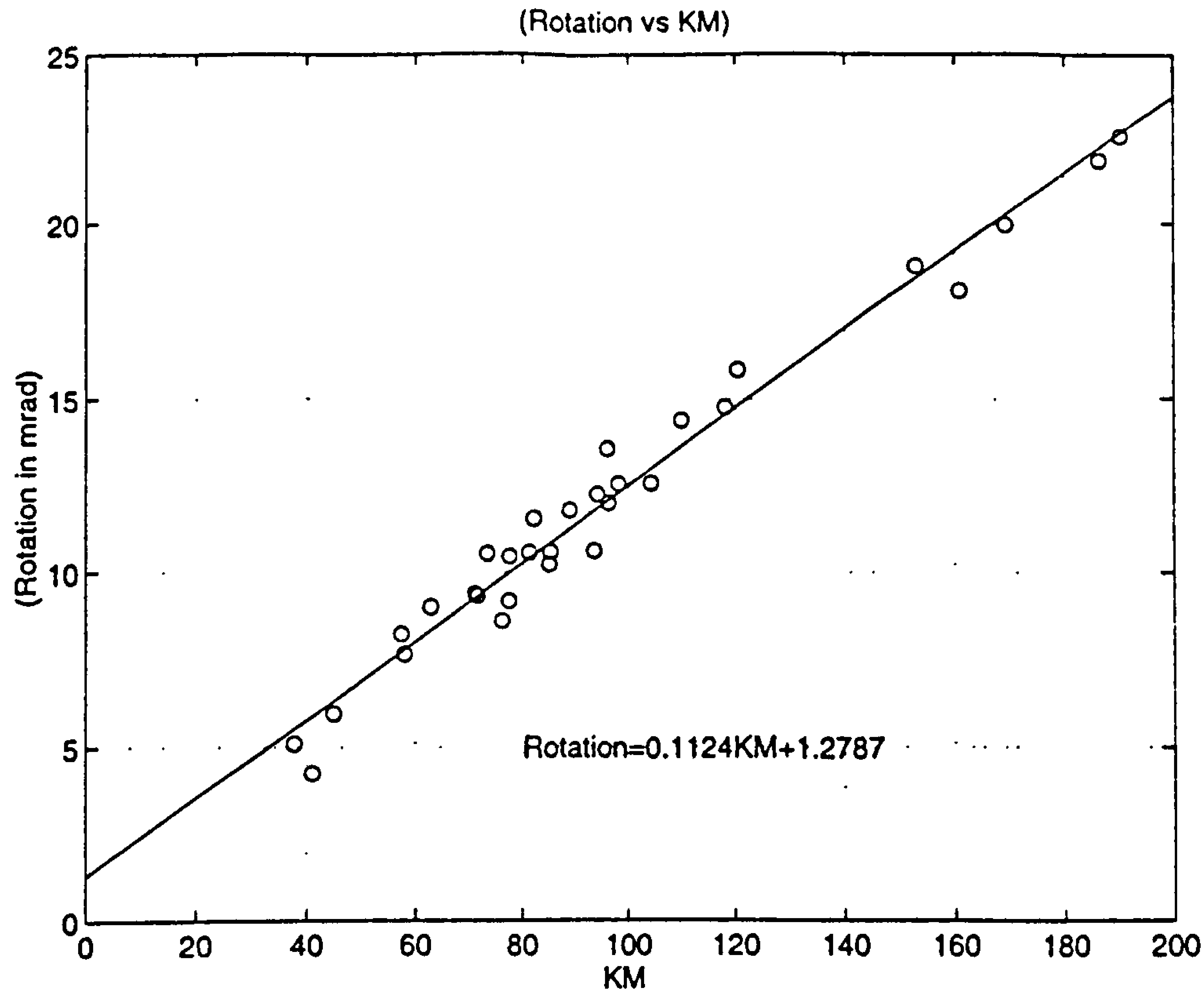

Fig. 4.11. Rotation versus KM.

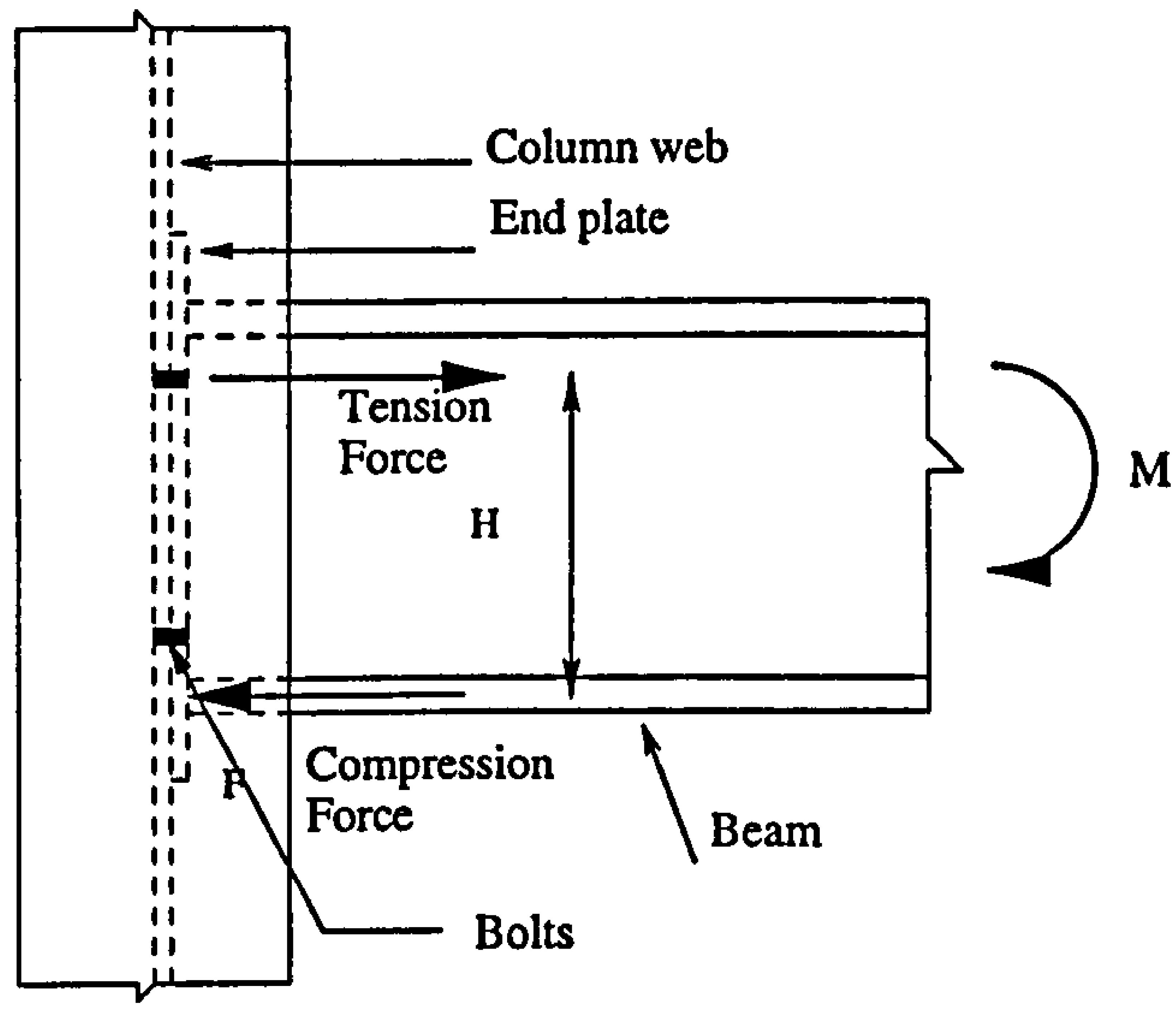

Fig 4.12(a) : Flush end plate connection to minor axis for external column

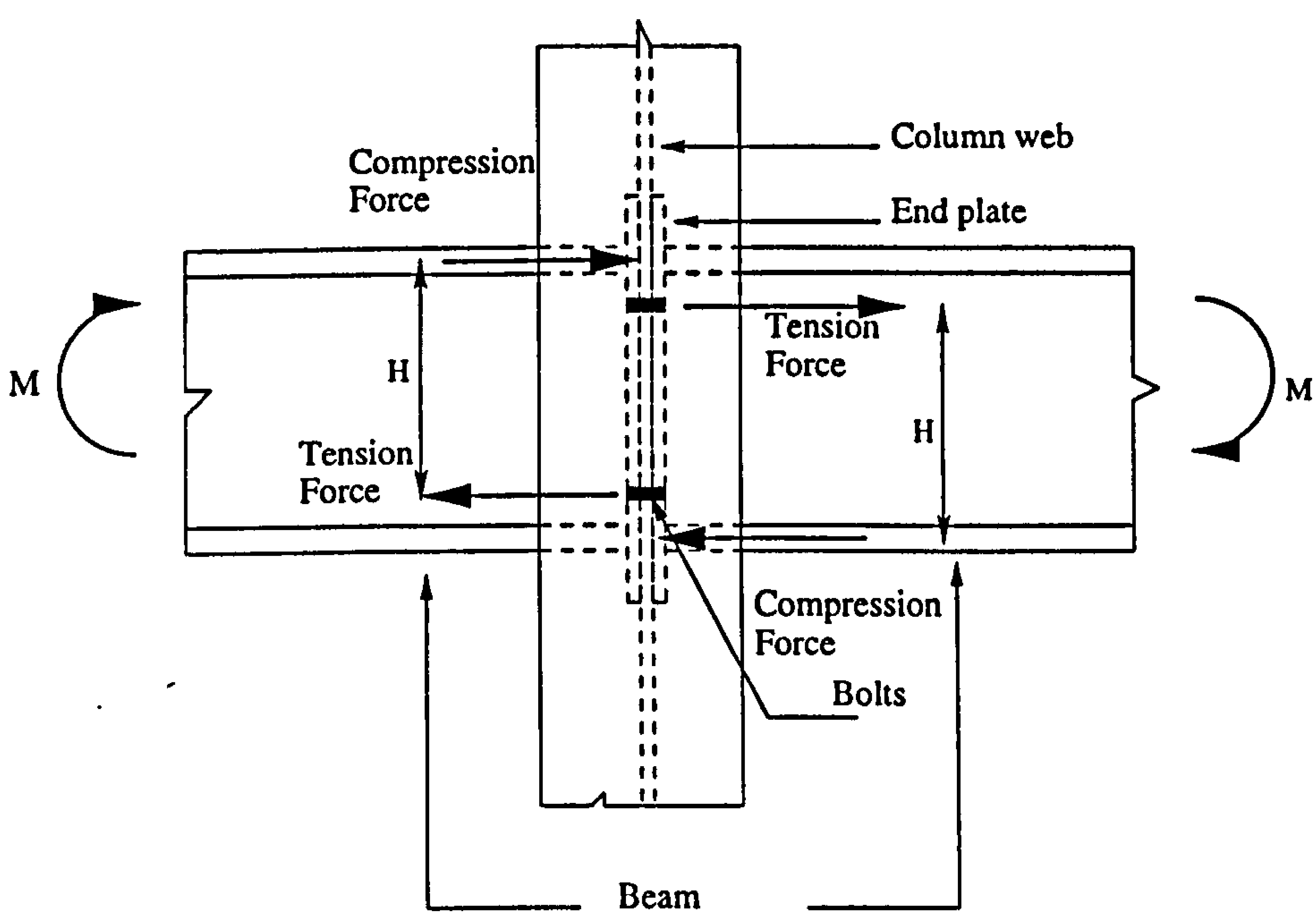

Fig 4.12(b) : Flush end plate connection to minor axis for internal column

Chapter 5

5.0 Design of composite beams in steel frames.

5.1 Introduction.

In multi-storey construction, steel beams and concrete slabs are frequently designed compositely. The two materials act compositely through inter-connection by means of headed studs or some other form of shear connection. The use of composite beams attracts designers because of the greater stiffness and higher load bearing capacity compared with their non-composite counterparts. This advantage may lead to a reduced section depth, a reduced height of the structure and through the use of profiled steel as permanent shuttering, to more rapid construction. Study shows that the savings in steel weight may reach up to 50% compared with non-composite beams[5.1].

The use of composite beams however has some disadvantages. Composite beams need shear connectors between the steel-concrete interface. The design of a composite beam is not as straightforward as that for non-composite beams.

5.2 Range of the study on composite beam steel frames.

As described in Chapter 1, composite beams are widely used in the U.K due to their advantages of greater stiffness and higher load bearing capacity. However, their advantages in unbraced frames may be limited due to formation of cracks resulting from hogging moment which may effect the performance of frames. The formation of cracks is discussed later in this chapter.

The advantages of composite beams in unbraced frames are also limited by the moment required by the connections. Steel frames designed for maximum wind load combined with minimum gravity load require substantial moment capacity at the connections. However, due to the beams being designed compositely, the sections selected reduce the beam depth significantly. As the moment capacity of the connections is dependent on the depth of the beams, reduction in beam depth will reduce the moment capacity of the connections. In this study, the moment capacity is selected from standard tables[5.2] in which the maximum moment capacity of the connections available is limited to the use of relatively thin flush and extended end plate connections. This is to ensure the ductility required to justify wind-moment design. In consequence, the size of the section may need to be increased so as to provide an adequate moment capacity in the connections. This increase in the beam section may result in it not being economical to design the beam compositely.

The frames studied ranged in height from two to four storeys, with the minimum number of bays taken as one and the maximum as four. Each bay was assumed to be of equal width. Both 6m and 9m bays were considered. The floor grid arrangement is shown in Figure 5.1 with a composite floor spanning 3 m. The following configurations of composite beam framing were therefore investigated:

- two-storey, one-bay
- four-storey, two-bay
- four-storey, four-bay.

The limitations on frame dimensions and loading conformed to those specified in the existing guide[5.3] for "wind-moment" design as shown in Figure 5.2. Frames were designed for combinations of maximum gravity load with minimum wind forces, and vice-versa.

5.3 Design methodology.

For ultimate limit states, the usual load combinations were applied, namely:

- Load case 1 - 1.4 (Dead) + 1.6 (Imposed)
- Load case 2 - 1.2 (Dead + Imposed + Wind)
- Load case 3 - 1.4 (Dead + Wind).

For deflection check under service load, the load combinations used were as follows:

- Load case 1 - 1.0 (Dead) + 1.0 (Imposed) + unfactored notional horizontal forces
- Load case 2 - 1.0 (Dead) + 0.8 (Imposed) + 0.8 (Wind)
- Load case 3 - 1.0 (Dead) + 1.0 (Wind).

Initially, the structure was designed as a bare frame for major-axis bending using the wind-moment software[5.3]. This provided beam sizes denoted as 'Section Designation I'. For minimum wind combined with gravity load theses are given in Table 5.1, and in Table 5.2 for maximum wind combined with minimum gravity load. The floor beams were then re-designed as composite beams under the vertical design loads, giving the sections listed in Table 5.3 and Table 5.4. These composite beams were designed based on a generic profile for sheeting as shown in Figure 5.3. The roof beam was not designed as a composite beam as the imposed load was quite small (1.5 kN/m^2). Kavianpour's software to analyse steel frames with semi-rigid connections[5.4] already described in Chapters 1 and 3, was modified to take into account the composite beam action and for hogging moment regions, the distance along the beam over which the concrete slab would be expected to crack. This software was used to check that the collapse load level of the frames was not lower than 1.0 times the factored loads at ULS and the horizontal deflection of the frames did not exceed a deflection index of 1/300 at SLS. Both ULS and SLS are checked by second order analysis.

5.3.1 Design of composite beams.

Composite beams are designed according to procedures described by Lawson[5.1]. To ease the task, Lawson has tabulated the beam's size according to types of concrete (normal or lightweight), beam spacing, grade of concrete, grade of steel, slab thickness, fire resistance, full or partial shear connection, and imposed load (distributed and point load). In this chapter, all composite beam sections were selected from these tables. Beams were selected for minimum depth. All steel sections were designed in S355 steel and for concrete of strength Grade 30. The beam sections were checked for maximum span with shear connectors placed in pairs per trough. By selecting shear connectors in pairs per trough, this increases the maximum possible span of a selected beam by providing full shear connection between steel and concrete in the composite beam. The design of composite beams is described according to the two frame types, namely:

1. for minimum wind combined with maximum gravity load
2. for maximum wind combined with minimum gravity load.

5.3.1.1 Minimum wind combined with maximum gravity load.

Table 24 as tabulated by Lawson[5.1] was used to design the composite beams based on normal weight concrete 135mm thick. To enable the proposed beam to be finally accepted, it needs to checked that the moment capacity of a standard connection[5.2] is capable of resisting the required moment determined by wind moment analysis as described earlier. Based on the proposed beam section, the moment resistance of the connection was selected from the standard tables[5.2]. The resulting beams are denoted as 'Section Designation II' in Table 5.3 which also gives the comparison of moment required and moment provided by the connections.

5.3.1.2 Design procedures for steel frames with composite beams.

The composite beams of 'Section Designation II' (Table 5.3) were reduced in size quite significantly compared with the bare steel beams of 'Section Designation I' (Table 5.1). Frames with 'Section Designation II' were then checked as follows:

1. Frames were analysed by a second order procedure[5.4] to check deflection does not exceed a sway index of 1/300 under service loading, with the combinations of unfactored loads shown in 5.3.

2. Frames were checked to ensure collapse was above the load level 1.0 for ULS under the combinations of factored loads shown in 5.3.
3. Account was taken in steps 1 and 2 of the reduced stiffness of cracked beams. The determination of cracked distance is described later in this chapter. In these analyses, the beam section was divided into cracked and uncracked regions. For the cracked section occurring in the region of hogging moment, the second moment of area was taken as that of the bare steel section, assuming no slab reinforcement. However, for uncracked sections the second moment of area was taken as that of the composite section. The determination of second moment of area for a composite beam is discussed later in the chapter.
4. Frames were reanalysed again as in step 3 until the cracked distances converged. This is achieved when the cracked distance for every beam no longer changes, which usually takes about 2 to 3 interations with tolerance of about 1%.
5. Frames were analysed for steps 1 to 4 with either the flexibility of the column web panel in shear included or excluded in the calculation of the initial stiffness of the connection. These analyses are needed so that the effects of their significance to the performance of the frames can be studied.

5.3.1.3 Determination of moment capacity of the composite beam.

The value of plastic moment capacity was determined from BS 5950 : Part 3: Section 3.1 : Appendix A[5.5] according to the resistance of the various elements of the beam. To determine the moment capacity of a composite beam the following formula was used:

$$M_c = M_s + k \times (M_{pc} - M_s)$$

where

M_s is the moment capacity of steel section ($p_y.S_{x-x}$),

k is the degree of shear connectors defined as:

$k = \frac{R_q}{R_s}$ for $R_s < R_c$ or $k = \frac{R_q}{R_c}$ for $R_c < R_s$, where R_q is total resistance of the shear connectors and R_c is total resistance of concrete flange.

M_{pc} is the plastic moment capacity of the composite beam for full shear connection calculated from Appendix B.2 in BS5950:Part 3[5.5]. The above formula showed that partial strength connection was taken into consideration in the calculation if appropriate. The calculated moment capacity of the beams was then limited to 90% in accordance to the wind-moment rules[5.3]. This reduction is necessary to provide restraint to the columns in accordance with clause 4.7.2 of BS 5950 Part 1[5.5]. It is also appropriate in order to avoid the very high rotation capacity demanded from the joints if the full plastic moment of a composite beam in sagging bending is to be attained[5.6].

5.3.1.4 Determination of second moment of area for uncracked section.

The value of second moment of area for the uncracked section was determined from BS 5950 : Part 3: Appendix B.3[5.5]. For a composite beam with equal flanges, the gross value of second moment of area I_g of the uncracked section is given as:

$$I_g = I_x + \frac{B_e (D_s - D_p)^3}{12 \times a_e} + \frac{A \times B_e (D_s - D_p)(D + D_s + D_p)^3}{4\{A \times \alpha_e + B_e (D_s - D_p)\}}$$

where

I_x is the second moment area of steel section,

B_e is the $L_z/8$ but not greater than B (BS 5950: Part 3),

D_s is the overall depth of the concrete flange,

D_p is the depth of the profile steel sheet,

A is the area of the steel beam,

D is the overall depth of the steel beam, and

α_e is modulus ratios defined as (E_a/E'_c) where E_a is the elastic modulus of steel taken as 210kN/mm^2 and E'_c is "effective" modulus of concrete. For buildings not intended for storing loads, the E'_c value may be taken equal to $E_{cm}/2$ where E_{cm} is the mean secant modulus for short term loading for normal weight concrete[5.7]. In this study α_e is therefore taken to be equal to $E_a/(0.5E_{cm})$. Details of calculations using the above formula are now presented.

Calculation for composite section properties

Sagging (Uncracked section)

$$I_g = I_x + \frac{B_e(D_s - D_p)^3}{12 \times a_e} + \frac{A_s \times B_e(D_s - D_p)(D + D_s + D_p)^3}{4\{A_s \times \alpha_e + B_e(D_s - D_p)\}}$$

For UB 254 x 102 x 25 (Beam span at 6 m)

$I_{x\text{-}x} = 3410\ cm^4$ $B_e = 2x600/8 = 150\ cm$ $D_s = 13.5\ cm$ $D_p = 5\ cm$

$A_s = 32.2\ cm^2$ $\alpha_e = 205\ 000/30\ 000 \times 0.5$ $D = 25.70\ cm$

$= 13.67$

$$I_g = 3410 + \frac{150(13.5-5)^3}{12\times13.67} + \frac{32.2\times150\times(13.5-5)(25.70+13.5+5)^2}{4\{32.2\times13.67+150(13.5-5)\}} = 15662\text{cm}^4$$

5.3.1.5 Determination of cracked distance.

The cracked distance was determined by limiting the tensile stress in the concrete to $0.15f_{cu}$ where f_{cu} is the cube strength of the concrete. This is based on a splitting tensile strength of $0.1f_{cu}$[5.8]. For flexure though, it is assumed that a modulus of rupture calculation is more appropriate. The modulus of rupture is calculated as M/Z where M is the bending moment at rupture and Z is the elastic modulus of the beam. The modulus of rupture is reported to be about 1.5 times the splitting cylinder strength[5.9]. This result in a limiting strength at the extreme fibre of $0.15f_{cu}$[5.8]. To test the sensitivity of the frame analyses to the assumptions concerning tensile strength, the two storey four bay frame CF2 (which showed the most deflection) was reanalysed with the limiting mean tensile strength f_{ctm} from Eurocode 4[5.7]. For concrete with f_{cu} = 30N/mm^2, this reduced the strength from 4.5N/mm^2 to 2.6N/mm^2. The results due to this change are discussed later in this chapter.

The value of maximum tensile stress was calculated from the following formula:

$$\sigma_{ct} = \frac{M_{hogging}\times y_e}{I_g}\times\frac{E_{cm}}{E_a}$$

where,

$M_{hogging}$ is the hogging moment at the support,

y_e is the distance from neutral axis to the top surface of concrete,

I_g is the uncracked moment of inertia of composite beam,

E_{cm} is the modulus of elasticity of concrete, and

E_a is the modulus of elasticity of steel.

Details of calculations are now presented for UB254x102x25 beam spanning 6m.

Determination of cracked distance in hogging bending:

$$\bar{y} = \frac{\sum Ay}{\sum A} = \frac{\left(A_s \times D/2\right) + \left(D_s - D_p\right)\frac{B_e}{\alpha}\left(D + \left(D_s - 0.5\left(D_s - D_p\right)\right)\right)}{A_s + \left(\frac{B_e}{\alpha} \times \left(D_s - D_p\right)\right)}$$

$$= \frac{\left(32.2 \times 25.7/2\right) + \left(8.5 \times 150/13.67\right)\left(25.7 + 13.5 - 0.5(13.5 - 5)\right)}{32.2 + \left(150 \times 8.5/13.67\right)} = 29.3\text{cm}$$

$$y_e = \left(D + D_s\right) - \bar{y} = 25.70 + 13.5 - 29.3 = 9.9\text{cm}$$

In this study, σ_{ct} taken not to exceed $0.15f_{cu}$. Hogging moment at leeward end (when connection reaches its limiting plastic resistance) equals to 5800 kN.cm. The tensile stress calculated at this moment is calculated below.

$$\sigma_{ct} = \frac{M_{hog} \times y_e}{I_g} \times \frac{E_{cm}}{E_s} = \frac{5800 \times 9.9}{15662} \times \frac{300}{2050} = 0.537\text{kN/cm}^2 = 5.37\text{N/mm}^2 > 0.15f_{ck}\left(4.5\text{N/mm}^2\right)$$

The calculated tensile stress showed that a crack distance needs to be calculated. To find the length of crack the following procedures were adopted:

1. Recalculate the hogging moment at increments of 1% of the beam span from the connection with the hogging moment.
2. This hogging moment was then used in the calculation of tensile stress which was then compared with the stress limit.

3. If the tensile stress was still greater than the stress limit, steps 1 and 2 were repeated again.
4. The length of crack was found when the final tensile stress was not greater than the stress limit.

The above procedures were programmed automatically to Kavianpour's software[5.4] by the present author.

To demonstrate that the beam spanning 9m was not cracked (as discussed later in 5.4.2), the following calculation is shown below.

For UB 356x171x51 (Beam span at 9 m)

$I_{x\text{-}x} = 14200\ \text{cm}^4$ $\quad B_e = 2\text{x}900/8 = 225\ \text{cm}$ $\quad D_s = 13.5\ \text{cm}$ $\quad D_p = 5\ \text{cm}$

$A_s = 64.6\ \text{cm}^2$ $\quad \alpha_e = 205\ 000/30\ 000 \times 0.5$ $\quad D = 35.56\ \text{cm}$

$= 13.67$

$$I_g = 14200 + \frac{225(13.5-5)^3}{12\times 13.67} + \frac{64.6\times 225\times(13.5-5)(35.56+13.5+5)^2}{4\{64.6\times 13.67+225(13.5-5)\}} = 47331\text{cm}^4$$

$$\bar{y} = \frac{\sum Ay}{\sum A} = \frac{\left(A_s \times D/2\right) + \left(D_s - D_p\right)\frac{B_e}{\alpha}\left(D + \left(D_s - 0.5\left(D_s - D_p\right)\right)\right)}{A_s + \left(\frac{B_e}{\alpha}\times\left(D_s - D_p\right)\right)}$$

$$= \frac{\left(64.6\times 35.56/2\right) + \left(8.5\times 225/13.67\right)\left(35.56+13.5-0.5(13.5-5)\right)}{64.6 + \left(225\times 8.5/13.67\right)} = 36.3\text{cm}$$

$$y_e = \left(D + D_s\right) - \bar{y} = 35.56 + 13.5 - 36.3 = 12.76\text{cm}$$

Hogging moment at right hand (when connection reached its limiting resistance) equals 8800 kN.cm.

$$\sigma_{ct} = \frac{M_{hog} \times y_e}{I_g} \times \frac{E_{cm}}{E_a} = \frac{8800 \times 12.76}{47331} \times \frac{300}{2050} = 0.35 kN/cm^2 = 3.5 N/mm^2 < 0.15 f_{ck}\left(4.5 kN/mm^2\right)$$

The calculated tensile shows prove that the beam was not cracked.

5.3.1.6 Connection stiffness for composite beam.

The stiffness used for the connection is determined from the following formulae due to Brown[5.9]:

without shear flexibility consideration

$$K_j = \frac{1}{\dfrac{0.7}{0.135 l_a^2}\left(\dfrac{m_{cf}^2}{t_{cf}^3} + \dfrac{m_{ep2}^3}{m_{ep1} t_{ep}^3}\right)}$$

with shear flexibility consideration

$$K_j = \frac{1}{\dfrac{0.7}{0.135 l_a^2}\left(\dfrac{m_{cf}^2}{t_{cf}^3} + \dfrac{m_{ep2}^3}{m_{ep1} t_{ep}^3} + \dfrac{\beta l_a}{0.38 Avc}\right)}$$

where the bending moment distribution factor 'β' is taken as 1.0 as suggested by Brown[5.9] for external columns. Initial stiffness with consideration of shear flexibility was only applied to external columns. This is because under minimum wind load and the influence of maximum gravity load, the shear effect on the internal columns is not regarded as significant.

5.3.1.7 Connection stiffness at Serviceability Limit State.

To judge the appropriate connection stiffness at SLS, a ratio between moment required and provided by the connection needs to be compared. The requirements for moment capacity at ULS shown in Table 5.1 are compared with moment provided by the standard tables[5.2]. For the weakest standard connection suitable for a 533x210UB, the moment resistance is 150kN.m, well above the required values (maximum 88kNm). When account is taken of the reduced demands for SLS loading, it is reasonable to assume that the joints will still be in the initial stiffness range. Although the weakest joint for a 406x140UB does not show such great excess strength (69kNm provided, 48kNm required at ULS), it is still regarded as reasonable to assume $S_{j,ini}$ for both ends of this beam as well.

5.3.1.8 Connection stiffness at Ultimate Limit States.

A stiffness equal to the initial value was used for windward connection and a stiffness equal to half the inital value was used for leeward connection. The reasons described in 3.3.4.2 are applicable here.

5.3.2 Maximum wind combined with minimum gravity load.

Table 23 as tabulated by Lawson[5.1] was used to design the composite beams based on lightweight concrete 125 mm thick. The results are listed in Table 5.4. The difference of slab depth by 10mm between normal and lightweight concrete results in the same fire resistance for the same strength grade of concrete. The lesser depth reduces the resistance of the composite section, but the dead load is also reduced. Thus the same section size results as previously designed for normal weight of 135mm thick. However, under maximum wind loading, it was found that these composite beams did not permit standard joints with adequate resistance to the wind moments. The required beam sections were therefore established from the moment provided by the standard connections[5.2] to meet the required moment calculated from wind moment analysis. The selection of joint details from the standard tables listed as page number[5.2] is shown in Table 5.4. The design of the beam with a deeper section therefore satisfied both the moment required from the beams and the moment required from the connections.

5.4 Parametric study.

The frame arrangements studied and the dimensions with loading are listed in Tables 5.1-5.3. Table 5.1 and Table 5.2, denoted as 'Section Designation I', concern wind moment designs for minimum wind combined with maximum gravity load and vice versa but neglecting composite action. Table 5.3, denoted as 'Section Designation II' showed the final design for frames from Table 5.1, accounting for composite design of the beams. Further changes in design of the beams governed by moment capacity of joints as explained in 5.2, are shown in Table 5.4.

5.4.1 Assessment of results.

The results are presented in two different ways as a result of the different design procedures due to different loading.

5.4.2 Results for minimum wind combined with maximum gravity load.

A comparison between frame CF1 and CF2 as in Table 5.1 shows a slight reduction in required moment of the connections in roof beams for the latter frame. This is because for frame CF1 (one bay frame), the required connection moment is governed by wind load, not dead plus imposed load. For beams spanning 9 m the results of moment required at connections for both roof and floor beams (Table 5.1) show the same value for the same floor level regardless of the number of bays. This is because the moment required at the connections is always governed by dead plus imposed load, not the wind load.

For frames designed for minimum wind combined with maximum gravity load, the results checked for collapse at ULS and for deflection limit at SLS showed that the frames can be designed with composite beams even though cracking affects the second moment area of beams in hogging bending. Results from Table 5.5 show the extent of cracking on beams measured from the leeward end of the beam. The frames were analysed with shear flexibility excluded from the initial stiffness of the connections. The results show that only floor beams in Frames CF2 and CF4 were cracked. This shows in these two frames the hogging moment tries to form a tensile stress which is greater than the tensile stress limit $0.15f_{cu}$. The frame identified as CF2 shows greater cracked lengths than frame CF4 because of a lesser number of bays which cause a higher hogging moment. No cracks developed for beam spans of 9 m because the tensile stress developed from the hogging moment at the limiting joint resistance was only 3.5 N/mm^2 compared with tensile stress limit for cracked of 4.5 N/mm^2($0.15f_{cu}$).

Results from Table 5.6 show the extent of cracking in frames analysed with shear flexibility included in the initial stiffness of external connections. The results show that the cracked length scarcely changed due to the lesser value of initial stiffness which results in less hogging moment. The formation of the plastic hinge at the joint however was not changed by shear flexibility of the connection and so at points at which such hinges formed it is to be expected that the moment at ULS was unchanged and therefore the cracking would be unchanged. Analysis showed all frames were stable at the ultimate limit state design load levels.

Results from Tables 5.7 and 5.8 show the frames were checked for deflection under serviceability loading, both with the initial stiffness of connection including shear flexibility and with shear flexibility excluded. The results show that the deflection increases slightly for frames with shear flexibility due to the less stiff connection. However, the results show overall sway deflections for the semi-rigid analyses with composite floor beams are less than the preferred sway index of height/300.

For the two bay four storey frame the analyses including shear deformation were repeated with the maximum tensile stress limited to 2.6N/mm^2. The frame still stands at a load level of 1.0 when analysed for ultimate limit state even though the cracked distances, shown in Table 5.6 as printed in parentheses ( ), slightly increase. This frame was also checked for serviceability loading to determine the change in the deflection. The results of deflection in Table 5.8 printed in parentheses ( ) show a change in the deflection index from 1/306 to 1/298, which is not significant. The value is just below the limit index of 1/300 but may be regarded as acceptable.

In this study, the shear connection was considered to be effective along the distance of L/2 as shown in Fig. 5.4(a) to resist longitudinal shear developed from the moment Mp at mid-span. However, due to the hogging moment developed at the end of the beam, this effective length is reduced to less than L/2 as shown in Fig. 5.4(b). In this study, this situation was assumed not to significantly affect the performance of the composite action of the beam as the hogging moment developed was very much lesser than the moment at mid-span.

5.4.3 Results for maximum wind combined with minimum gravity load.

Frames designed for these loads were not checked by Kavianpour's analysis for ULS and SLS. Table 5.4 shows the beam sizes governed by (i) moment capacity required from the beam and (ii) the beam size which results in adequate joint resistance. It was found that for almost all frames the beam size had to be increased substantially to provide an adequate joint. For this reason, no further analyses were undertaken. For frames with final design sections of greater than or equal to the size of bare steel section it is not economical to design the beam compositely.

5.5 Conclusions.

Despite the assumption that a composite beam has greater stiffness and moment capacity, a straightforward application of composite beam design does not always result in frames of adequate overall safety and stability. This has been shown on frames in which maximum wind loads combined with minimum gravity load governed the design requirements. The standard connections[5.2] do not provide enough moment resistance in such cases when account is taken of the reduced size of composite beam. It is concluded that the use of composite beams in sway frames is appropriate for the low wind load relative to gravity load case in low rise frames up to four storeys. To quantify the relative loading, it is proposed that wind-moment design be permitted with composite beams provided that the wind moment does not cause any increase in the size of the steel section beyond that required to resist the gravity load. To justify the safety of this, the most critical frame CF2 was re-analysed as a limiting case with the moment resistance of the joints set to equal the design wind-moment. The frame was found to be still stable at the ULS design load level.

References

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Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Floor LL	Roof DL	Roof LL	Basic Wind Speed (m/s)	Ground Roughness Factor	Section Designation (I) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Connection Requirements Bending Moment (kNm) Floor	Bending Moment Roof	Shear Force (kN) Floor	Shear Force Roof
2 Storey 1 Bay	CF 1	6.0 (composite beam at 3m span)											1st 406x140x46	305x102x25	Upto 2nd Storey	203x203x46	N/A	1st 40	14	1st 173	69
4 Storey 2 Bay	CF 2												1st 406x140x46 2nd 406x140x46 3rd 406x140x46	305x102x25	Upto 2nd Storey 2nd-4th Storey	203x203x60 152x152x37	254x254x89 203x203x46	1st 48 2nd 38 3rd 32	12	1st 178 2nd 175 3rd 173	69
2 Storey 4 Bay	CF3												1st 406x140x46	305x102x25	Upto 2nd Storey	203x203x46	203x203x46	1st 32	12	1st 173	69
4 Storey 4 Bay	CF 4												1st 406x140x46 2nd 406x140x46 3rd 406x140x46	305x102x25	Upto 2nd Storey 2nd-4th Storey	203x203x60 152x152x37	254x254x89 203x203x46	1st 46 2nd 37 3rd 31	12	1st 173 2nd 175 3rd 178	69
			4.5	3.5	2	6.0	5.0	7.5	3.75	1.5	37	4									
2 Storey 1 Bay	CF 5	9.0 (composite beam at 3m span)											1st 533x210x82	406x140x39	Upto 2nd Storey	203x203x52	N/A	1st 68	25	1st 259	104
4 Storey 2 Bay	CF 6												1st 533x210x82 2nd 533x210x82 3rd 533x210x82	406x140x39	Upto 2nd Storey 2nd-4th Storey	254x254x73 203x203x46	305x305x118 203x203x52	1st 88 2nd 75 3rd 66	25	1st 263 2nd 260 3rd 258	104
2 Storey 4 Bay	CF 7												1st 533x210x82	406x140x39	Upto 2nd Storey	203x203x52	203x203x71	1st 68	25	1st 259	104
4 Storey 4 Bay	CF 8												1st 533x210x82 2nd 533x210x82 3rd 533x210x82	406x140x39	Upto 2nd Storey 2nd-4th Storey	254x254x73 203x203x46	305x305x118 203x203x52	1st 88 2nd 75 3rd 66	25	1st 263 2nd 260 3rd 258	104

Table 5.1 Wind-moment design considering minimum wind in conjunction with maximum gravity load

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m ^{2}) Floor DL	Floor LL	Roof DL	Roof LL	Basic Wind Speed (m/s)	Ground Roughness Factor	Section Designation (I) Universal Beam Floor	Universal Beam Roof		Universal Columns External	Universal Columns Internal	Connection Requirements Bending Moment (kNm) Floor	Bending Moment (kNm) Roof	Shear Force (kN) Floor	Shear Force (kN) Roof
2 Storey 1 Bay	CF 9	6.0 (composite beam at 3m span)											1st 406x140x39	305x102x25	Upto 2nd Storey	254x254x73	N/A	1st 209	65	1st 139	73
4 Storey 2 Bay	CF 10												1st 457x152x52 2nd 406x140x39 3rd 356x127x33	305x102x25	Upto 2nd Storey 2nd-4th Storey	305x305x97 203x203x46	305x305x118 203x203x71	1st 325 2nd 214 3rd 127	45	1st 172 2nd 140 3rd 115	69
2 Storey 4 Bay	CF 11												1st 356x127x33	305x102x25	Upto 2nd Storey	203x203x46	203x203x71	1st 63	23	1st 104	69
4 Storey 4 Bay	CF 12												1st 356x127x33 2nd 356x127x33 3rd 356x127x33	305x102x25	Upto 2nd Storey 2nd-4th Storey	203x203x46 254x254x73	203x203x52 305x305x97	1st 173 2nd 115 3rd 70	27	1st 129 2nd 112 3rd 104	69
			6.0	5.0	2	6.0	3.5	4.0	3.75	1.5	52	1									
2 Storey 1 Bay	CF 13	9.0 (composite beam at 3m span)											1st 457x152x52	406x140x39	Upto 2nd Storey	254x254x73	N/A	1st 228	77	1st 162	104
4 Storey 2 Bay	CF 14												1st 457x152x52 2nd 457x152x52 3rd 457x152x52	406x140x39	Upto 2nd Storey 2nd-4th Storey	305x305x97 203x203x60	356x368x129 254x254x73	1st 333 2nd 222 3rd 135	54	1st 182 2nd 161 3rd 155	104
2 Storey 4 Bay	CF 15												1st 457x152x52	406x140x39	Upto 2nd Storey	203x203x52	254x254x73	1st 71	33	1st 155	104
4 Storey 4 Bay	CF 16												1st 457x152x52 2nd 457x152x52 3rd 457x152x52	406x140x39	Upto 2nd Storey 2nd-4th Storey	254x254x89 203x203x46	305x305x97 203x203x71	1st 182 2nd 124 3rd 81	36	1st 159 2nd 156 3rd 155	104

Table 5.2 Wind-moment design considering maximum wind in conjunction with minimum gravity load

Basic Frame Type	Frame Identification	Width of Bay (m)	Height of Column Ground (m)	Height of Column Elevated (m)	No. of Longitudinal Bays (m)	Width of Longitudinal Bays (m)	Gravity Load (kN/m 2) Floor DL	Floor LL	Roof DL	Roof LL	Section Designation (II) Floor Composite beam	Roof Bare steel section		Universal Columns External	Internal	Connection Requirements Bending Moment (kNm) Floor	Roof	Shear Force (kN) Floor	Roof	Connection Provided Bending moment (kN.m) Floor	Roof
2 Storey 1 Bay	CF 1										1st 254x102x25	305x102x25	Upto 2nd Storey	203x203x46	N/A	1st 40	14	1st 173	69	40	50
4 Storey 2 Bay	CF 2	6.0 (composite beam at 3m span)									1st 254x102x25 2nd 254x102x25 3rd 254x102x25	305x102x25	Upto 2nd Storey 2nd-4th Storey	203x203x60 152x152x37	254x254x89 203x203x46	1st 48 2nd 38 3rd 32	12	1st 178 2nd 175 3rd 173	69	58 58 37	50
2 Storey 4 Bay	CF3										1st 254x102x25	305x102x25	Upto 2nd Storey	203x203x46	203x203x46	1st 32	12	1st 173	69	40	50
4 Storey 4 Bay	CF 4										1st 254x102x25 2nd 254x102x25 3rd 254x102x25	305x102x25	Upto 2nd Storey 2nd-4th Storey	203x203x60 152x152x37	254x254x89 203x203x46	1st 46 2nd 37 3rd 31	12	1st 173 2nd 175 3rd 178	69	58 58 37	47
			4.5	3.5	2	6.0	5.0	7.5	3.75	1.5											
2 Storey 1 Bay	CF 5										1st 356x171x51	406x140x39	Upto 2nd Storey	203x203x52	N/A	1st 68	25	1st 259	104	85	69
4 Storey 2 Bay	CF 6										1st 356x171x51 2nd 356x171x51 3rd 356x171x51	406x140x39	Upto 2nd Storey 2nd-4th Storey	254x254x73 203x203x46	305x305x118 203x203x52	1st 88 2nd 75 3rd 66	25	1st 263 2nd 260 3rd 258	104	88 88 75	69
2 Storey 4 Bay	CF 7	9.0 (composite beam at 3m span)									1st 356x171x51	406x140x39	Upto 2nd Storey	203x203x52	203x203x71	1st 68	25	1st 259	104	85	69
4 Storey 4 Bay	CF 8										1st 356x171x51 2nd 356x171x51 3rd 356x171x51	406x140x39	Upto 2nd Storey 2nd-4th Storey	254x254x73 203x203x46	305x305x118 203x203x52	1st 88 2nd 75 3rd 66	25	1st 263 2nd 260 3rd 258	104	88 88 85	69

Table 5.3 Frames designed for composite beams by considering minimum wind in conjunction with maximum gravity load

Basic Frame Type	*Width of* Bay (m)	*Beam's* position	*Bare steel beam section*	*Composite beam section* (vertical load)	*Beam section governed by wind moment* on beam	on joint	*Selected joint detail*	*Moment* required (kN.m)	*Moment* provided (kN.m)
2 Storey 1 Bay	6	Roof	305x102x25	N/A	N/A	N/A	P 208	65	92
		1st Floor	406x140x39	254x102x25	406x140x39	457x152 UB	P 215	209	211
4 Storey 2 Bay	6	Roof	305x102x25	N/A	N/A	N/A	P205	45	50
		3rd Floor	356x127x33	254x102x25	356x127x33	406x140 UB	P210	127	143
		2nd Floor	406x140x39	254x102x25	406x140x39	533x210 UB	P210	214	220
		1st Floor	457x152x52	254x102x25	457x152x52	686x254 UB	P214	325	326
2 Storey 4 Bay	6	Roof	305x102x25	N/A	N/A	N/A	P205	23	50
		1st Floor	406x140x39	254x102x25	254x102x25	254x102 UB	P208	63	71
4 Storey 4 Bay	6	Roof	305x102x25	N/A	N/A	N/A	P205	27	50
		3rd Floor	356x127x33	254x102x25	254x102x25	254x102 UB	P205	70	71
		2nd Floor	356x127x33	254x102x25	254x102x28	356x127 UB	P215	115	157
		1st Floor	356x127x33	254x102x25	356x127x33	406x140 UB	P215	173	180
2 Storey 1 Bay	9	Roof	406x140x39	N/A	N/A	N/A	P206	77	102
		1st Floor	457x152x52	254x146x37	356x127x39	457x191 UB	P216	228	230
4 Storey 2 Bay	9	Roof	406x140x39	N/A	N/A	N/A	P205	54	69
		3rd Floor	457x152x52	254x146x37	254x146x37	406x140 UB	P210	135	155
		2nd Floor	457x152x52	254x146x37	356x127x39	457x191 UB	P216	222	230
		1st Floor	457x152x52	254x146x37	457x152x52	533x210 UB	P217	333	342
2 Storey 4 Bay	9	Roof	406x140x39	N/A	N/A	N/A	P205	33	69
		1st Floor	457x152x52	254x146x37	254x146x37	254x146 UB	P208	71	72
4 Storey 4 Bay	9	Roof	406x140x39	N/A	N/A	N/A	P205	36	69
		3rd Floor	457x152x52	254x146x37	254x146x37	305x102 UB	P208	81	86
		2nd Floor	457x152x52	254x146x37	254x146x37	306x127 UB	P215	124	139
		1st Floor	457x152x52	254x146x37	306x127x37	406x140 UB	P215	182	186

Table 5.4 Frames with composite beam checked for connection resistance (Maximum wind combined with minimum gravity load)

Basic Frame Type	Frame Identification	Width of Bay (m)	Location of crack on beam from right hand side (in cm)																										
			Design moment (kN.m)	Moment capacity (kN.m)	Location of beam	Ultimate Limit State												Service Limit State											
						Bay 1			Bay 2			Bay 3			Bay 4			Bay 1			Bay 2			Bay 3			Bay 4		
						LC1	LC2	LC3	LC1	LC2	LC3	LC1	LC2	LC3	LC1	LC2	LC3	LC1	LC2	LC3	LC1	LC2	LC3	LC1	LC2	LC3	LC1	LC2	LC3
2 Storey 1 Bay	CF 1	6.0 (composite beam at 3m span)	230.9	240.8	Floor	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4 Storey 2 Bay	CF 2				3rd floor	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A
					2nd floor	12	12	NC	12	6	NC	N/A	N/A	N/A	N/A	N/A	N/A	6	6	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A
					1st floor	12	12	NC	12	6	NC	N/A	N/A	N/A	N/A	N/A	N/A	6	6	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A
2 Storey 4 Bay	CF3				Floor	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
4 Storey 4 Bay	CF 4				3rd floor	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
					2nd floor	12	6	6	12	6	6	12	6	6	6	NC	NC	6	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
					1st floor	12	NC	6	12	NC	6	12	NC	6	6	NC	NC	6	NC	NC	6	NC	NC	6	NC	NC	NC	NC	NC
2 Storey 1 Bay	CF 5	9.0 (composite beam at 3m span)	519.4	567.6	Floor	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4 Storey 2 Bay	CF 6				3rd floor	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A
					2nd floor	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A
					1st floor	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A
2 Storey 4 Bay	CF 7				Floor	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
4 Storey 4 Bay	CF 8				3rd floor	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
					2nd floor	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
					1st floor	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC

Table 5.5 Frames designed with composite floor beams by considering minimum wind in conjunction with maximum gravity load
(Shear flexibility not included)

Basic Frame Type	Frame Identification	Width of Bay (m)	Design moment (kN.m)	Moment capacity (kN.m)	Location of beam	Location of crack on beam from right hand side (in cm)																							
						Ultimate Limit State												Service Limit State											
						Bay 1			Bay 2			Bay 3			Bay 4			Bay 1			Bay 2			Bay 3			Bay 4		
						LC1	LC2	LC3	LC1	LC2	LC3	LC1	LC2	LC3	LC1	LC2	LC3	LC1	LC2	LC3	LC1	LC2	LC3	LC1	LC2	LC3	LC1	LC2	LC3
2 Storey 1 Bay	CF 1	6.0 (composite beam at 3m span)	230.9	240.8	Floor	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4 Storey 2 Bay	CF 2				3rd floor	(6)	(6)	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A
					2nd floor	12(36)	12(42)	0(30)	12(30)	6(30)	0(6)	N/A	N/A	N/A	N/A	N/A	N/A	6(30)	6(36)	NC	0(12)	0(18)	NC	N/A	N/A	N/A	N/A	N/A	N/A
					1st floor	12(36)	12(42)	0(30)	12(30)	(24)	0(6)	N/A	N/A	N/A	N/A	N/A	N/A	6(36)	6(36)	NC	0(18)	0(18)	NC	N/A	N/A	N/A	N/A	N/A	N/A
2 Storey 4 Bay	CF3				Floor	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
4 Storey 4 Bay	CF 4				3rd floor	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
					2nd floor	12	6	6	12	6	6	12	6	6	NC	NC	NC	6	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
					1st floor	12	NC	6	12	NC	6	12	NC	6	NC	NC	NC	6	NC	NC	6	NC	NC	6	NC	NC	NC	NC	NC
2 Storey 1 Bay	CF 5	9.0 (composite beam at 3m span)	519.4	567.6	Floor	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4 Storey 2 Bay	CF 6				3rd floor	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A
					2nd floor	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A
					1st floor	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A	NC	NC	NC	NC	NC	NC	N/A	N/A	N/A	N/A	N/A	N/A
2 Storey 4 Bay	CF 7				Floor	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
4 Storey 4 Bay	CF 8				3rd floor	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
					2nd floor	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC
					1st floor	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC	NC

Table 5.6 Frames designed with composite floor beams by considering minimum wind in conjunction with maximum gravity load
(Shear flexibility included)

Basic Frame Type	Frame Identification	Width of Bay (m)	Section Designation Use	Load Case	Deflection check (2nd order analysis) Uncracked	Deflection check (2nd order analysis) Cracked	Location	Connection Provided Bending moment (kN.m) Floor	Initial Stiffness Prediction (kNm/mrad) External Column	Initial Stiffness Prediction (kNm/mrad) Internal Column	Selected joint detail
2 Storey 1 Bay	CF 1	6.0 (composite beam at 3m span)	Section Designation (II) (Semi-Rigid Connection)	LC1	1/1301	N/A	Roof	50	5.48	N/A	P205
				LC2	1/437	N/A	Floor	40	3.30	N/A	P205
				LC3	1/370	N/A					
4 Storey 2 Bay	CF 2			LC1	1/558	1/549	Roof	47	4.92	5.48	P205
				LC2	1/354	1/351	3rd floor	37	3.35	3.30	P205
				LC3	1/326	N/A	2nd floor	58	6.83	8.36	P212
							1st floor	58	6.83	8.36	P212
2 Storey 4 Bay	CF3			LC1	1/944	N/A	Roof	50	5.48	5.48	P205
				LC2	1/1143	N/A	Floor	40	3.30	3.30	P205
				LC3	1/1017	N/A					
4 Storey 4 Bay	CF 4			LC1	1/579	1/568	Roof	47	4.92	5.48	P205
				LC2	1/694	N/A	3rd floor	37	3.35	3.30	P205
				LC3	1/648	N/A	2nd floor	58	6.83	8.36	P212
							1st floor	58	6.83	8.36	P212
2 Storey 1 Bay	CF 5	9.0 (composite beam at 3m span)		LC1	1/3256	N/A	Roof	69	11.34	N/A	P205
				LC2	1/804	N/A	Floor	85	14.24	N/A	P212
				LC3	1/684	N/A					
4 Storey 2 Bay	CF 6			LC1	1/641	N/A	Roof	69	11.25	11.34	P205
				LC2	1/712	N/A	3rd floor	75	14.07	14.24	P212
				LC3	1/761	N/A	2nd floor	88	14.24	20.98	P212
							1st floor	88	14.24	20.98	P212
2 Storey 4 Bay	CF 7			LC1	1/957	N/A	Roof	69	11.34	14.01	P205
				LC2	1/2454	N/A	Floor	85	14.24	20.98	P212
				LC3	1/2286	N/A					
4 Storey 4 Bay	CF 8			LC1	1/564	N/A	Roof	69	11.25	11.34	P205
				LC2	1/1364	N/A	3rd floor	75	14.07	14.24	P212
				LC3	1/1442	N/A	2nd floor	88	14.24	20.98	P212
							1st floor	88	14.24	20.98	P212

Table 5.7 Frames designed for composite beams by considering minimum wind in conjunction with maximum gravity load
(Shear flexibility not included)

Basic Frame Type	Frame Identification	Width of Bay (m)	Section Designation Use	Load Case	Deflection check (2nd order analysis) Uncracked	Cracked	Location	Connection Provided Bending moment (kN.m) Floor	Initial Stiffness Prediction (kNm/mrad) External Column	Internal Column	Selected joint detail
2 Storey 1 Bay	CF 1	6.0 (composite beam at 3m span)	Section Designation (II) (Semi-Rigid Connection)	LC1	1/1255	N/A	Roof	50	4.53	N/A	P205
				LC2	1/420	N/A	Floor	40	3.19	N/A	P205
				LC3	1/357	N/A					
4 Storey 2 Bay	CF 2			LC1	1/516	1/500(1/445	Roof	47	4.06	5.48	P205
				LC2	1/329	1/322(1/298	3rd floor	37	2.83	3.30	P205
				LC3	1/306	0(1/298)	2nd floor	58	5.47	8.36	P212
							1st floor	58	5.47	8.36	P212
2 Storey 4 Bay	CF3			LC1	1/937	N/A	Roof	50	4.53	5.48	P205
				LC2	1/1136	N/A	Floor	40	3.19	3.30	P205
				LC3	1/1986	N/A					
4 Storey 4 Bay	CF 4			LC1	1/560	1/540	Roof	47	4.06	5.48	P205
				LC2	1/670	N/A	3rd floor	37	2.83	3.30	P205
				LC3	1/630	N/A	2nd floor	58	5.47	8.36	P212
							1st floor	58	5.47	8.36	P212
2 Storey 1 Bay	CF 5	9.0 (composite beam at 3m span)		LC1	1/2867	N/A	Roof	69	8.85	N/A	P205
				LC2	1/702	N/A	Floor	85	10.10	N/A	P212
				LC3	1/597	N/A					
4 Storey 2 Bay	CF 6			LC1	1/603	N/A	Roof	69	10.07	11.34	P205
				LC2	1/668	N/A	3rd floor	75	9.78	14.24	P212
				LC3	1/723	N/A	2nd floor	88	12.87	20.98	P212
							1st floor	88	12.87	20.98	P212
2 Storey 4 Bay	CF 7			LC1	1/920	N/A	Roof	69	8.85	14.01	P205
				LC2	1/2353	N/A	Floor	85	10.10	20.98	P212
				LC3	1/2222	N/A					
4 Storey 4 Bay	CF 8			LC1	1/545	N/A	Roof	69	10.07	11.34	P205
				LC2	1/1316	N/A	3rd floor	75	9.78	14.24	P212
				LC3	1/1415	N/A	2nd floor	88	12.87	20.98	P212
							1st floor	88	12.87	20.98	P212

Table 5.8 Frames designed for composite beams by considering minimum wind in conjunction with maximum gravity load
(Shear flexibility included)

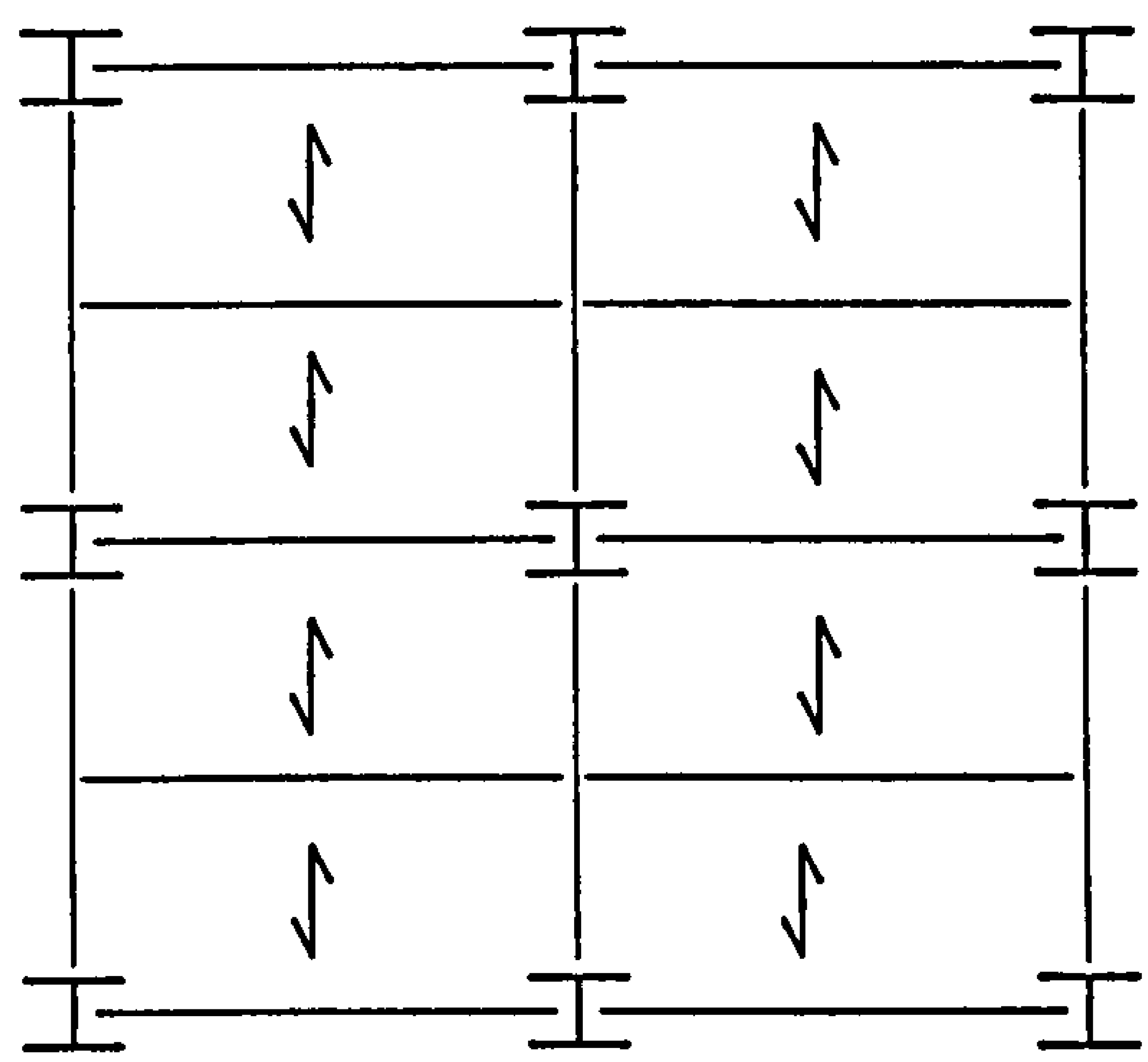

Fig 5.1 Floor grids arrangement for composite beam design

	Minimum wind	Maximum wind
Number of storeys	2	8
Number of bays	2	4
Bay width	6.0	9
Storey height (bottom storey)	6.0	6
Storey height (elsewhere)	3.5	5
Dead load on floors	3.50 kN/m^2	5.00 kN/m^2
Imposed load on floors	4.00 kN/m^2	7.50 kN/m^2
Dead load on roof	3.75 kN/m^2	3.75 kN/m^2
Imposed load on roof	1.50 kN/m^2	1.50 kN/m^2
Basic wind speed	37 m/s	52 m/s
Grade steel	S355	S355

Fig 5.2 Range of "wind-moment" study on major axis

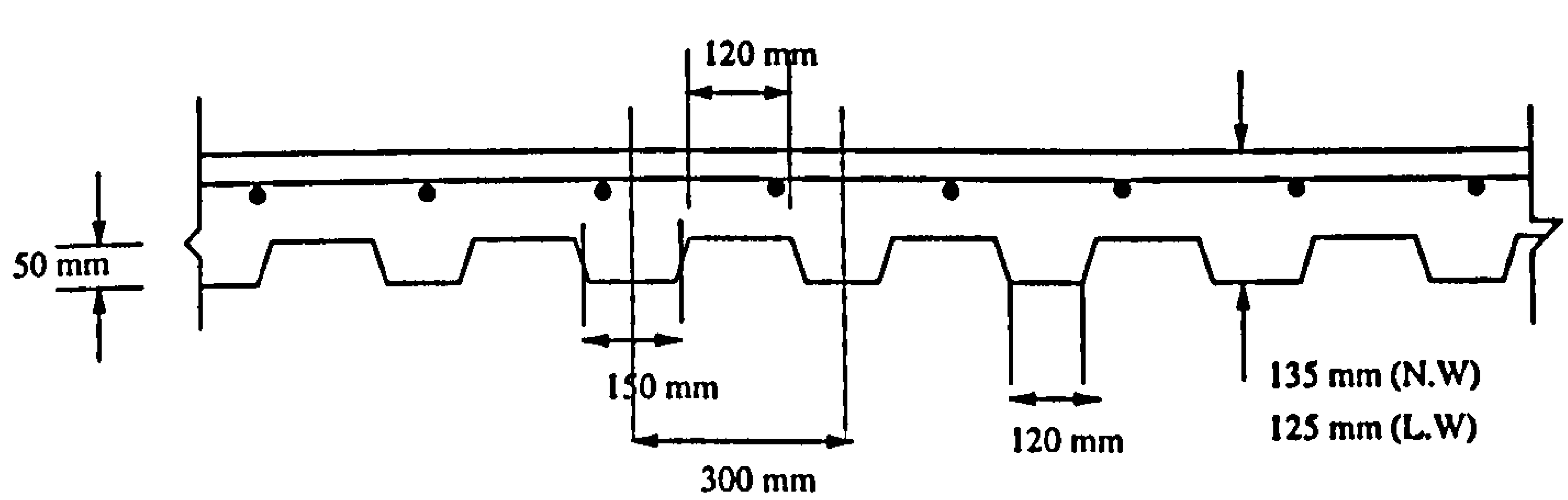

Fig 5.3 Cross section through generic profile concrete slab

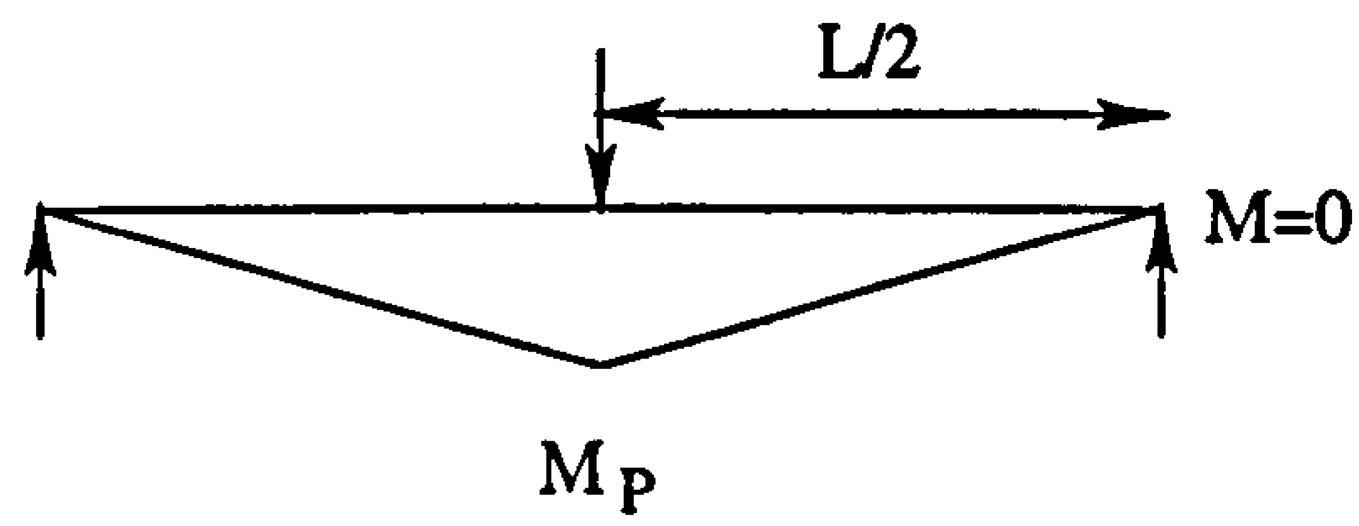

Fig. 5.4 (a) Design of shear connection for simple supported beam

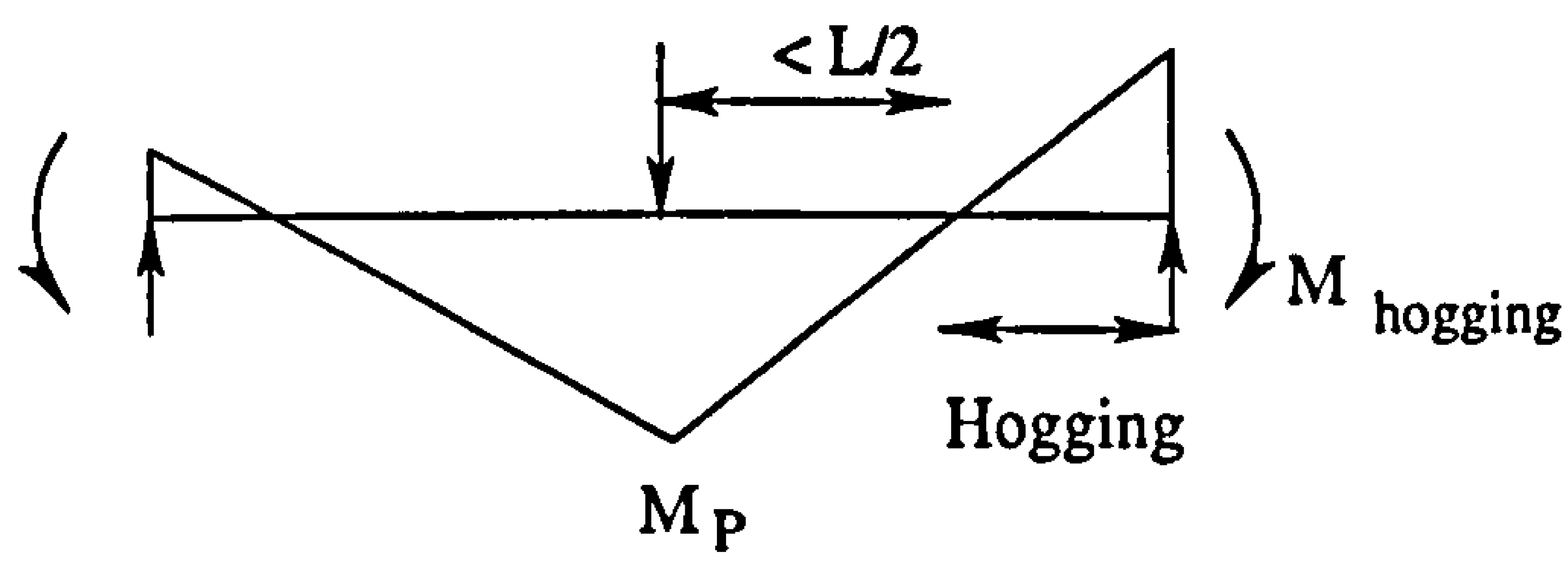

Fig. 5.4 (b) Design of shear connection for beam with hogging moment

Chapter 6

6.0 Push tests for composite steel-concrete beams with pin-connected shear stud.

6.1 Introduction.

This chapter describes a series of push tests designed to investigate a shear connector (Fig. 6.1) for composite steel-concrete beams. The connector has been developed by Pneutek Ltd[6.1]. The system is reported to be the fastest and the most economical method for installing studs to steel roof and floor decking in composite construction[6.1]. The system is five to ten times faster than welding, screwing, or using powder actuated tools which reduces the fastening cost[6.1]. These advantages would make a significant further contribution to the popularity of composite construction. Each stud is connected to a base plate that has two circular holes specially designed to locate fastening pins. The pin is fired into the steel section with a pneumatic pinning machine fed by an air compressor. Under normal operating conditions the pressure would be 1200 kN/m^2. The aims of the push tests were to determine the shear strength, ductility, and failure modes of the shear connectors. The shear strength and ductility of the studs are observed by plotting the load-slip curve. Failure may occur due to crushing of concrete, shearing or pull-out of the fastening pins, and the deformation of base plates or profiled decking. The tests are divided into three series, with each having different characteristics of shear stud. Seven specimens have been tested and their definition is given in Table 6.1.

Chapter 6

6.0 Push tests for composite steel-concrete beams with pin-connected shear stud.

6.1 Introduction.

This chapter describes a series of push tests designed to investigate a shear connector (Fig. 6.1) for composite steel-concrete beams. The connector has been developed by Pneutek Ltd[6.1]. The system is reported to be the fastest and the most economical method for installing studs to steel roof and floor decking in composite construction[6.1]. The system is five to ten times faster than welding, screwing, or using powder actuated tools which reduces the fastening cost[6.1]. These advantages would make a significant further contribution to the popularity of composite construction. Each stud is connected to a base plate that has two circular holes specially designed to locate fastening pins. The pin is fired into the steel section with a pneumatic pinning machine fed by an air compressor. Under normal operating conditions the pressure would be 1200 kN/m^2. The aims of the push tests were to determine the shear strength, ductility, and failure modes of the shear connectors. The shear strength and ductility of the studs are observed by plotting the load-slip curve. Failure may occur due to crushing of concrete, shearing or pull-out of the fastening pins, and the deformation of base plates or profiled decking. The tests are divided into three series, with each having different characteristics of shear stud. Seven specimens have been tested and their definition is given in Table 6.1.

Five specimens were prepared using a solid concrete slab and two specimens were prepared with PMF metal decking. The specimens were labelled according to the type and the number of specimen tested. For example, S30-1 represents test one on a solid slab with grade 30 concrete. The specimens labelled as S30-1, S30-2, S30-3, CF30-4, and CF30-5 were the first part of the tests. The second and third parts of the tests were single specimens labelled as S30-6 and S30-7 respectively.

6.2 Summary of previous tests.

The use of compressed air-actuated steel fasteners was introduced by Pneutek in 1976[6.2]. Tests on the stud system were conducted to check the penetration ability of the fastener, resistance to pull-out and uplift. The results concluded that the fasteners possess adequate resistance to pullout and uplift forces for steel decks ranged from 0.64mm to 1.29mm. The fasteners used by Pneutek were later tested for cyclic loading[6.3] on a roof deck. The results concluded that Pneutek fasteners perform adequately in cyclic loading provided that they are not installed with an excessive force which may damage the surface of the roof deck. The research was continued on the strength characteristics of Pneutek pins subjected to direct shear, tensile tests, and full scale diaphragm shear tests[6.4] on steel deck. It was reported that in direct shear tests, the strength was limited by contact area over the pin diameter and the sheet thickness. For tensile tests, the axial tension is very much dependent on the materials and the driving conditions. For the full scale diaphragm tests, a slight reduction in strength was recorded compared with the predicted strength due to difficulties in driving the pins. The Pneutek pins system was further tested on steel decks for roof construction[6.5] to determine if they met the requirements of the Factory Mutual Research Corporation[6.6] for securing steel roof decking to structural steel supports. The test results indicated that the Pneutek air actuated fasteners did meet the requirements. Tests on the use of the Pneutek studs system for composite steel-concrete beams are described in this chapter.

6.3 Preparation of specimens.

6.3.1 Specimens arrangements and configurations.

Details of the test specimens are given in Table 6.1 and their geometries are shown in Figures 6.2 to 6.7. Both specimens, CF30-4 and CF30-5, with profiled metal decking had the troughs positioned perpendicular to the direction of loading. The basic configurations of the seven specimens were recommended by the Steel Construction Institute and are similar to that recommended in EC4 [6.7]. In each slab one layer of 200 x 200 A142 6 mm diameter mesh was placed with 15 mm concrete cover. In specimen S30-1 a second layer was placed as shown in Figure 6.2. Four specimens had the base plate of the connectors parallel to the web of the steel section, so that the fastening pins would be fully embedded into the web of the steel section. Specimens CF30-5, S30-6, and S30-7 had the base plate orientation perpendicular to the web as shown in Figure 6.5 to 6.7. In the case of CF30-5 the base plates had to be positioned perpendicular to the web because the shape of the profiled decking prevented them from being placed parallel. Specimen CF30-4 (Fig. 6.4) had the base plate positioned paralled to the column web. A 30 mm lateral spacing (Fig. 6.4) for the centre stud in slab A was necessary due to the shape of the profiled decking. Details of the geometry for CF51 and CF70 profiled deckings are shown in Figure 6.8.

6.3.2 Manufacture and mechanical properties of studs.

Studs were delivered to the University of Warwick laboratory with the base plate welded to one end of the shank of the stud. Dimensions for the connectors as used in Part 1, 2, and 3 of the programme are given in Figures 6.1, 6.9 and 6.10 respectivly. The diameter of the stud was measured to be 14.7 mm and the height to be 101 mm (from the top to the face of the base plate) for the first six specimens. However, for specimens S30-7, the diameter of stud was reduced to 12.7mm as shown in Figure 6.10.

The base plate has two holes designed to allow the two fastening pins to penetrate into the section. The two holes are specially designed with an upward protrusion as shown in Figure 6.1. This shape allows material from the section to fill in the space beneath the plate when the pin penetrates, permitting close contact between the base plate and the steel section.

For S30-6, the upward protrusion of the holes was flattened before installation. The flattening was achieved by using the Pneutek air-compressor machine to fire to the holes without a fastening pin being present. This is done to reduce the force needed later to install the pin on the steel beam and thus increase the grip of the pin.

The stud system was changed for the test on specimen S30-7. Each of the studs had a flat base plate of dimensions 32.5 mm by 62.7 mm by 4.7 mm thick. This new stud system therefore had geometric differences to the one used in tests No. 1 to 6. The new base plate (Figure 6.10) was thicker by 0.5 mm and the diameter of the stud was reduced from 14.7 mm to 12.7 mm. The reason for these changes was to reduce the resistance of the stud itself such that failure was in the concrete and not, as in the previous six tests, due to the fastening pins. The two holes in the base plate, to locate the pins, had a small chamfer at the surface (grooved by about 1 mm as shown in Fig 6.10) in contact with the beam, and this feature was to promote the required level of fixture. The mechanical properties of the material in the stud system and the method of welding the stud to the base plate were the same as in the first six tests. The weld of the stud to the base plate is assumed to have enough shear strength to resist the same shear force as the full diameter of the stud.

A series of tensile tests have been carried out to determine the tensile strength and the failure modes of the stud system. The tensile tests were of two types (Fig. 6.11). The first type consisted of two studs connected together at their base plates by 6 mm mild steel bolts. The second type consisted of two specimens with two studs, one connected to each of the two beam flanges using the fastening pins. The specimen was placed in an Amsler tensile machine and slowly pulled apart until there was a failure of a pin either in tension or due to pull-out. Tensile action on shear connectors may be neglected in the design of composite members if it is less than 10% of the design shear resistance of the stud system[6.7]. The results for these tests are given in Table 6.2. Prior to the push tests, it was expected that the fastening pin had sufficient pull-out strength so as not to be critical in a push test. The shear strength of a stud connector is directly proportional to its cross sectional area and it is expected to deform provided the pin and base plate combination are adequate to resist the required shearing force. However, the actual shear strength of a stud connector embedded into a concrete slab is influenced by the compressive strength and the elastic modulus of concrete [6.8].

6.3.3 Deck pins.

The fastening pins have been specially designed for penetration into structural steel. They are made from heat-treated carbon steel with very high ultimate tensile strength (> 1500 N/mm^2). The fastening pins for the Pneutek system were supplied together with a plastic holder and coated with stainless steel or fluoropolymer for maximum corrosion resistance. The pins used in this test series were batch No. K66075. The geometry of a fastening pin is shown in Figure 6.1, 6.9, 6.10. For the sixth and seventh tests(S30-6 and S30-7), new fastening pins were used. These had been specially designed to improve their performance from that of the pins used in the previous tests. The new pins were longer and the length of the point was shorter. These changes increased the overall penetration depth by about 4mm. The air pressure was set to a value in the range 1340 to 1380 kN/m^2 before firing took place. The fixing of the pins was performed by Bartrum of Pneutek. The small size of the specimens, having a tendency to move when a pin impacted, was considered by Bartrum to be the reason why the pressure had to be increased. Even after taking this action to promote a successful fixture, there was concern that the fixing was not of the quality expected. The pin is considered to be installed correctly by inspecting the plastic holder; it should split from the pin, and the pin head should be in close contact with the base plate.

For specimen S30-7, the new stud system were also fixed to the beam by Bartrum, using the standard gun but with air pressure of 1413kN/m^2. The pins were secured successfully, with full penetration observed by the pin's pointed-end protruding several millimetres out of the far side of the flange.

6.3.4 Casting.

All push specimens were cast with concrete designed to Grade 30. The concrete mixture was designed by adopting a method proposed by the Department of Environment[6.9]. Ordinary Portland cement was used for the normal weight concrete mix together with coarse aggregate of size around 10 mm and fine aggregate graded as medium sand. The expected concrete cube strength during testing was 30 N/mm^2, for age 28 days after casting. However, experience shows that normal concrete mixes are frequently over-strength. Therefore, a trial mix was prepared for Grade 25 concrete and its cube strength was measured after age 7 days. An expected concrete strength can be predicted for any age henceforth using the 7 days trial mix cube strength. The trial mix had an average concrete strength, at age 7 days, of 30.0 N/mm^2, and was therefore considered to be satisfactory. For the sixth push test, S30-6, the average concrete cube strength was recorded at 25 N/mm^2, for an age of 14 days at the test day. For the seventh push test, S30-7, the average concrete cube strength was recorded at 38 N/mm^2, for an age of 7 days at the test day. A mistake in measuring the amount of cement for this test meant that the concrete gained strength more quickly than it should have.

Each specimen had a slab A and a slab B. Slab A was cast one day earlier than slab B. Both slabs were cast horizontally and compacted using a vibrator to eliminate air pockets around the studs. Seven batches of concrete were prepared for each set of seven slabs to be cast. Two 100 mm cubes were cast from each batch for the purpose of determining the cube strength on the day of testing. Several additional cubes were available to determine how long it would be before the strength was in the desired range of 30-35 N/mm^2. After casting, the specimens and cubes were kept together under wet hessian for curing.

6.3.5 Test procedure.

The test rig and a specimen are shown in Figure 6.12. The slabs were positioned vertically and bedded to the strong floor using dental paste. The spreader beam was placed onto the ends of the beam flanges and then bolted together to hold the beam section in position so as to apply the load uniformly. A ball joint of capacity 80 tonnes, was placed above the centre of the web. Load was applied through a manually operated hydraulic jack and monitored with a pre-calibrated 100 tonne capacity load cell. Predicted values for the ultimate resistance per stud, P_u, can be determined by equation 6.4 of Eurocode 4[6.7], and are:

$$P_u = 0.7 \times \left(\frac{\pi \times d^2}{4}\right) f_u = 0.7 \times \left(\frac{\pi \times 14.7^2}{4}\right) \times 450 \times 10^{-3} = 53.5\text{kN} \text{(for 14.7 diameter stud)}$$

$$P_u = 0.7 \times \left(\frac{\pi \times d^2}{4} \right) f_u = 0.7 \times \left(\frac{\pi \times 12.7^2}{4} \right) \times 450 \times 10^{-3} = 39.9\text{kN (for 12.7 diameter stud)}$$

where d is the shank diameter, and f_u is the UTS of the stud, assumed to be 450 N/mm^2.

The failure load may be influenced by the grade of concrete, type of decking, number of studs, and the method of stud connection. Here, however, the pin fasteners can also influence the failure mode(s) and the ultimate resistance of the stud system.

6.3.6 Instrumentation.

To measure displacement four displacement transducers were used as shown in Figure 6.12. Two of the transducers were placed at the base of beam's web, one on each side. Vertical displacement or slip was monitored by these transducers and was taken as the mean value. Another two transducers were placed 100mm from the top of each slab to measure the lateral displacement between the steel and concrete. All transducers were connected to an Orion data logger, and had been calibrated before the test series. A levelling instrument was used to level all transducers in order to read the best possible vertical and lateral displacements.

6.3.7 Measurement and loading sequence.

The data logger system was set-up to read displacements in millimetres and load in kN. All transducer readings were then initialised to zero. The specimen was first loaded with at least 30 kN to settle the specimen in the rig. After waiting for about 2 minutes, the specimen was then unload to zero. All transducer readings were then initialised again to zero, ready for the test. The following load sequence was generally used. An increment of 30 kN, or 5 kN per stud, was applied to the specimen. Readings were recorded after two minutes had elapsed. This time elapse allowed the specimen to reach an equilibrium state. The incremental load procedure was then repeated until either there was a significant increase in slip or there was a sound of a pin fracturing. A slip increase of 0.2 mm or a fracturing pin was considered significant. The loading on the specimen was then controlled in slip increments of 0.2 mm. Readings were recorded after a three minute interval. The test was continued until ultimate failure, when both slabs became completely detached from the beam.

6.4 Results and discussion.

6.4.1 Load-slip curves.

Load per stud is plotted against slip for the seven specimens as shown in Figures 6.13 to 6.19. The overall results of the load-slip curves are summarized in Table 6.3. The specimen failure modes are summarized in Table 6.4. It should be noted that the results are based on each stud being assumed to resist an equal share of the applied load. At ultimate failure all specimens have a recorded slip value in the range 3 mm to 7 mm. The results show that the new form of shear connection has a less ductile behaviour than conventional welded shear studs. This is due to failure of the pins before any plastic deformation of the stud can be mobilised. Both specimens using PMF steel decking gave a higher initial stiffness when compared to the solid slab specimens. This is due to the presence of profiled decking (1.2mm thick) which inhibits the movement of the pins compared to the solid concrete slab. The maximum resistance for solid slab specimens S30-1 to S30-3 was recorded at between 42 and 46 kN per stud. Slight differences in the results may come from a slight difference in the concrete cube strength (Table 6.3) or the penetration depth of the fastening pins (Table 6.5). Specimen S30-1 (Figure 6.1) with two layers of A142 mesh gave no significant additional resistance.

For specimens S30-6 the maximum resistance was recorded as 56 kN per stud. This showed an improvement of about 20% compared with the test configurations S30-1 to S30-3. However, a small slip at ultimate failure of 3.1 mm and the sudden catastrophic failure of all pins in slab B showed that the stud system is not ductile. Figure 6.18 indicates the load-slip values when pins were heard to fracture. The second and last fracturing of pins occurred at ultimate resistance and afterwards there was separation of slab B from the steel section. All pins in slab B had fractured in shear at the same time. Slab A was still connected to the steel section with a lateral separation of about 0.4 mm. Overall the test has shown a significant improvement in maximum resistance of the stud system. However, there is still a need to change its design if the ductility of the stud is to be mobilised.

The load per stud (kN) against slip (mm) for specimen S30-7 is plotted in Figure 6.19. The shape of the load-slip curve showed that a stud connector had a maximum shear resistance of 43.2 kN. This resistance is based on the assumption that all studs and pins transmitted the same force; it is believed that a number of the pins had failed before the maximum load had been attained (see Figure 6.19). The low slip at maximum load of 0.8 mm, and the sudden fracture of pins from a load of 34.3 kN per stud, showed that the stud system had little ductility. The falling branch part of the load-slip curve decreases rapidly with slip. This does not indicate that the resistance of the six studs is likewise decreasing because as the slip increases and pins continually fracture an unknown number of the studs become inactive. When the slip was 5.8 mm the last pin fractured and one of the slabs detached itself from the beam. For 12.7 mm diameter stud, the shear resistance determined from Eurocode 4, is P_u = 39.9 kN. However, this is based on an assumed strength; the actual value of ultimate tensile strength for the steel is not known to the author. The stud system also possess extra shear resistance due to the base plate and this can be assumed to have a value calculated as length and thickness of base plate (Fig. 6.10) multiplied by the cube strength of the concrete. The result of calculation is shown below.

$$62.7 \times 4.7 x f_{cu} = 294.7 \times 38.7 = 11.4 \text{kN}.$$

The maximum shear resistance of 43.2kN per stud from the test is lower than the assumed value of combined resistance of the stud and its base plate calculated as 39.9 + 11.4 = 51.3kN. There is a possibility that the pins' shear resistance is lower than 51.3kN which results in their early fracture failure. If the UTS of the pins is assumed to have a higher value (i.e. 2000 N/mm^2), the pins' shear resistance should be sufficient to mobilise gross section yielding at the base of the 12.7 mm diameter stud.

It is not a straightforward task to compare the test result of 43.2 kN per stud with the shear resistance calculated above. One reason is the assumption made on the UTS of the steels. A second reason is that the test result does not take into account pins fracturing which may be due to load not shared equally and this means that the measured shear resistance of 43.2 kN per stud is on the low side. Further evidence to support the case against making a reliable comparison is obtained by observing the local deformation of a pin and of the base plate and flange either side of the interface between them. The pin is of a steel that has a hardness greater than the steel of the components it connects. Under the influence of the shear force the pin elongates the holes in the base plate and flange. The pin deforms such that it 'kinks' in this local region. The pins must therefore experience combined shear and flexural deformations as shown in Fig. 6.20, not pure shear failure. The size of the flexural effect is unknown and cannot be determined theoretically.

The maximum resistance of the specimens with decking were lower than for the solid slabs. This is due to the thickness of the decking(1.2 mm) which reduced the actual penetration depth of the fastening pins into the steel beam. The pins used for CF51 specimen were 1 mm longer than the rest of the specimens. Specimen CF30-4(Figure 6.3) recorded maximum resistance of 42 kN per stud and specimen CF30-5 of 34 kN per stud, respectively. There is a possibility that the latter has a lower value due to the base plate being positioned perpendicularly to beam's web. At maximum resistance, the lateral displacements were in the range 0.24 to 0.41 mm as shown in Table 6.3 and these were not visible at this stage in the test.

6.4.2 Failure modes.

There were no cracks on the outside surface of the concrete in any of the specimens tested. However, there was a slight crushing of concrete around the base plates, penetrating to the top of the upturned lips on the plates. No profiled metal decking specimens failed due to rib punching or tearing. There was a lot of deformation around the holes of the base plate (Fig 6.20(d)). The deformation was due to forces transmitted by the fastening pins and was one reason why the maximum load per stud was less than the theoretical value of 53.5 kN (this being based on shear resistance of the stud). During testing there were 7 to 10 loud bangs which indicated that a pin had fractured or a pin had pulled-out of the section. The load-slip when such failures were heard are given in Figures 6.13 to 6.19. The maximum resistance of the connection came after 2 or 3 of these fractures had occurred. When the load reached its maximum value the lateral displacement at the top of the slabs started to develop, and soon became visible. When a slab finally separated from the steel section, all 6 pins had failed either by facturing at the head/shank junction or by sliding out of its hole. One of the stud connectors was removed from the concrete by breaking out the surrounding concrete to examine the system for weld and stud deformation. It was found that the friction-type weld used to join the stud to the base plate had not failed and the stud had not deformed. However, inspection of the base plate showed that there was significant plastic deformation in the region where a pin was placed and that this deformation must have caused the top of the pin to experience a complex and indeterminate stress state. Visual inspection of the slabs after each test showed that most of the six pins had fractured as described, but that often there was one pin that had failed by pull-out. It is expected that, if perfectly installed, the pins should have had sufficient resistance to prevent them from being pulled-out. However, the pull-out failure can occur due to insufficient pin penetration into the steel beam.

One difficulty with substantiating the behaviour proposed here is that the visual evidence of pin failure is only available at the end of the test, and this does not necessary show the condition at the instant a pin failed. However, what can be seen is the local deformation described, together with pin fractures several millimetres into both the base plate and flange. The author believes that most of the recorded slip, until pins fail, was due to this local pin deformation and that the contribution from movement of the stud was small.

6.5 Comparison results with other researchers.

A new shear stud connector proposed by Spit[6.10] has been reported (Fig. 6.21). It uses the same method as proposed by Pneutek but with different geometrical characteristics of the shear stud connector. The proposed Spit connector has a stud diameter measured at 12 mm and height measured at 101 mm as shown in Figure 6.21. The assumed design value of the stud alone according to EC4[6.7] is equal to 35.6kN calculated as below.

$$P_u = 0.7\times\left(\frac{\pi\times d^2}{4}\right)f_u = 0.7\times\left(\frac{\pi\times 12.0^2}{4}\right)\times 450\times 10^{-3} = 35.6\text{kN}$$

However, the design value is expected to be higher if combined with resistance due to the stud support. A special steel sheeting which supports the stud has a yield strength of 235 N/mm^2 and thickness of 2mm (Fig. 6.21) and was used instead of base plate. The support was welded to the stud as shown in Fig 6.21 which improves the shear resistance of the stud system by increasing the area of resistance. The description of the experimental tests was almost identical with those of the author except that the loading was cycled 25 times between 5 and 40% of the expected failure load before the final load increments were imposed. The result showed that a good ductile behaviour has been recorded in all of the push-out tests. The load-slip curve was plotted with maximum resistance of 57 kN per stud as shown in Figure 6.22. Only a few horizontal cracks on the outside surfaces of concrete slab were reported.

6.6 Conclusions.

The results of the 5 push tests showed that the fastening pins failed in most cases by fracture, and that this mode of failure occurred before the shear stud had yielded. This mode of failure was in part due to the shape and deformation of the shear stud's base plate. The strength and the ductility of the connection may be improved by modifying the pin and the base plate. This may be done by increasing the penetration depth of the pins or by modifying the properties of the base plate. In addition, the method of installing the pins should be optimised in order to mobilise the stud. Maximum resistances obtained showed that the base plates gave encouraging results by positioning them parallel to the column web. It was found that the specimen with CF51 decking has a better maximum resistance than that with CF70. However, this result could be the result of the pins being 1 mm shorter in the latter specimen (and in the three solid specimens). No cracks were found at the outermost surface of the concrete slab and this observation confirms that the stud did not deform much. The total slip of 4 to 7 mm was therefore associated with deformation of the base plate-pin-steel beam fixture. Significantly, none of the base plates sheared from its stud, indicating that the weld was not the weak link in these tests.

For the stud system S30-7, it may be concluded that the system had not increased the shear resistance or ductility. No improvement has been made when compared with the old stud system with identical pins used in test S30-6. In reality the shear resistance is less because pins started to fail at a lower load per stud. From the limited test data available the author cannot provide an explanation for why the shear resistance of S30-7 is lower. Overall the seven tests showed that the stud system needs to be improved so that it has a better ductility and maximum resistance. Further analysis, including comparisons with theoretical models and other experiments, need to be performed.

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Table 6.1: Test specimens

Test No.	Report Label	Studs per slab	Lateral spacing	Layer of mesh	Decking	Orientation of plate to col. web
1	S30-1	3	0	2	None	parallel
2	S30-2	3	0	1	None	parallel
3	S30-3	3	0	1	None	parallel
4	CF30-4	3	30*	1	CF51	parallel
5	CF30-5	3	0	1	CF70	perpendicular
6	S30-6	3	0	1	None	perpendicular
7	S30-7	3	0	1	None	perpendicular

Note:- (*) only for slab A)

Table 6.2: Mechanical properties of studs

Type of connection	Failure load (kN)	Description of mode of failure
(bolted)	19.62 kN	Plate bent and one bolt broken
(bolted)	19.13 kN	Plate bent and one bolt broken
(pinned)	21.78 kN	One pin broken and one pin pulled out
(pinned)	18.15 kN	One pin broken and one pin pulled out

Table 6.3: Overall results of load/slip curves

Specimen	Maximum resistance per stud (kN)	Slip at maximum resistance (mm)	Lateral displacement at maximum resistance (mm)	Initial load/slip slope (kN/mm)	Applied load per stud at first sound of failure (kN)
S30-1	42.5	1.63	N/A	67	30
S30-2	42.0	2.04	0.41	64	39
S30-3	46.8	1.98	0.34	78	42
CF30-4	42.2	0.79	0.24	154	30
CF30-5	34.5	1.35	0.34	154	30
S30-6	55.8	3.1	0.26	125	50
S30-7	42.3	0.78	N/A	111	34

Table 6.4: Failure modes

Specimens	Position of stud	Failure modes				
		Pins		Base plates	Concrete	Profiled decking
		Slab A	Slab B	Slabs A and B	Slabs A and B	Slabs A and B
S30-1	Top Centre Bottom	fracture fracture fracture	pull fracture fracture	slightly deformed	slightly crushed around the base plates	N/A
S30-2	Top Centre Bottom	fracture fracture fracture	fracture pull fracture	slightly deformed	slightly crushed around the base plates	N/A
S30-3	Top Centre Bottom	pull fracture fracture	shear fracture fracture	slightly deformed	slightly crushed around the base plates	N/A
CF30-4	Top Centre Bottom	fracture fracture fracture	pull fracture fracture	N/A	N/A	slightly deformed
CF30-5	Top Centre Bottom	fracture fracture fracture	fracture fracture fracture	N/A	N/A	slightly deformed
S30-6	Top Centre Bottom	fracture fracture fracture	fracture fracture fracture	slightly deformed	slightly crushed around the base plates	N/A
S30-7	Top Centre Bottom	fracture fracture fracture	fracture fracture fracture	slightly deformed one of the base plate slightly bent	slightly crushed around the base plates	N/A

Table 6.5 Height of studs and pins in solid slabs and metal deck specimens

Specimen	Height of studs in mm (top of stud to base of deck)						Height of deck pins in mm (top of pin to base of deck)												Orientation of plate to web of steel section.
	Slab A			Slab B			Slab A						Slab B						
							Top		Centre		Bottom		Top		Centre		Bottom		
	Top	Centre	Bottom	Top	Centre	Bottom	a	b	a	b	a	b	a	b	a	b	a	b	
S 30 -1	105.1	105.6	105.6	105.3	104.7	105.4	8.8	8.5	8.6	8.5	8.6	8.4	8.9	8.8	8.8	8.8	9.0	9.0	Parallel
S 30-2	104.7	105.3	104.9	105.5	105.5	105.3	8.7	8.3	8.5	8.5	8.7	9.5	8.6	8.3	8.9	8.9	8.8	9.6	Parallel
S 30-3	105.5	105.2	105.7	105.5	105.6	104.9	8.7	7.6	9.0	8.6	8.5	8.7	8.9	9.0	8.7	8.8	8.7	9.0	Parallel
CF 30-4	104.5	105.5	105.3	105.6	105.4	105.7	8.9	10.2	8.9	9.3	9.0	8.4	9.4	9.0	9.0	8.9	8.9	8.7	Parallel
CF 30-5	105.0	105.4	105.6	105.6	105.7	105.6	8.6	7.8	8.2	8.4	8.2	8.2	8.6	8.4	8.4	9.1	8.7	9.1	Perpendicular

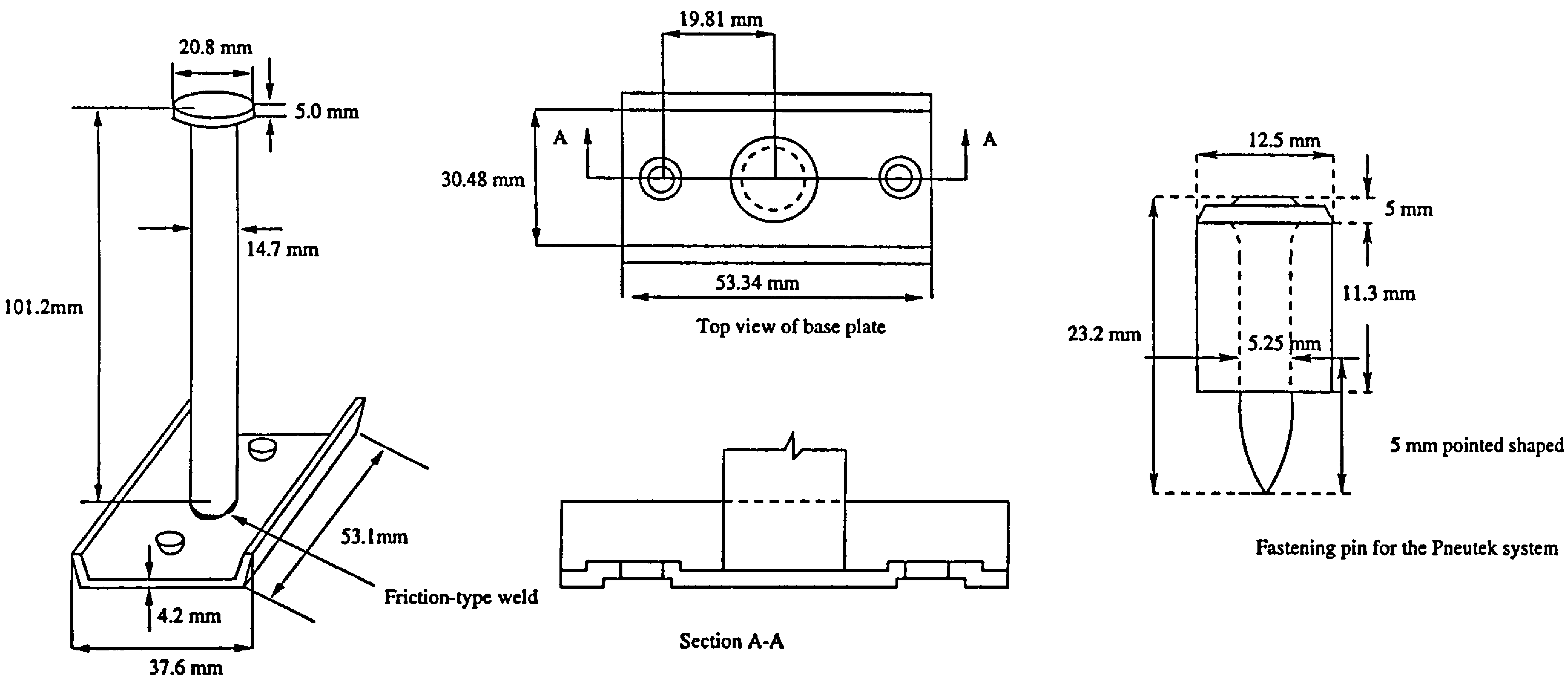


Figure 6.1: Geometrical characteristics of the Pneutek air-safe shear stud connector

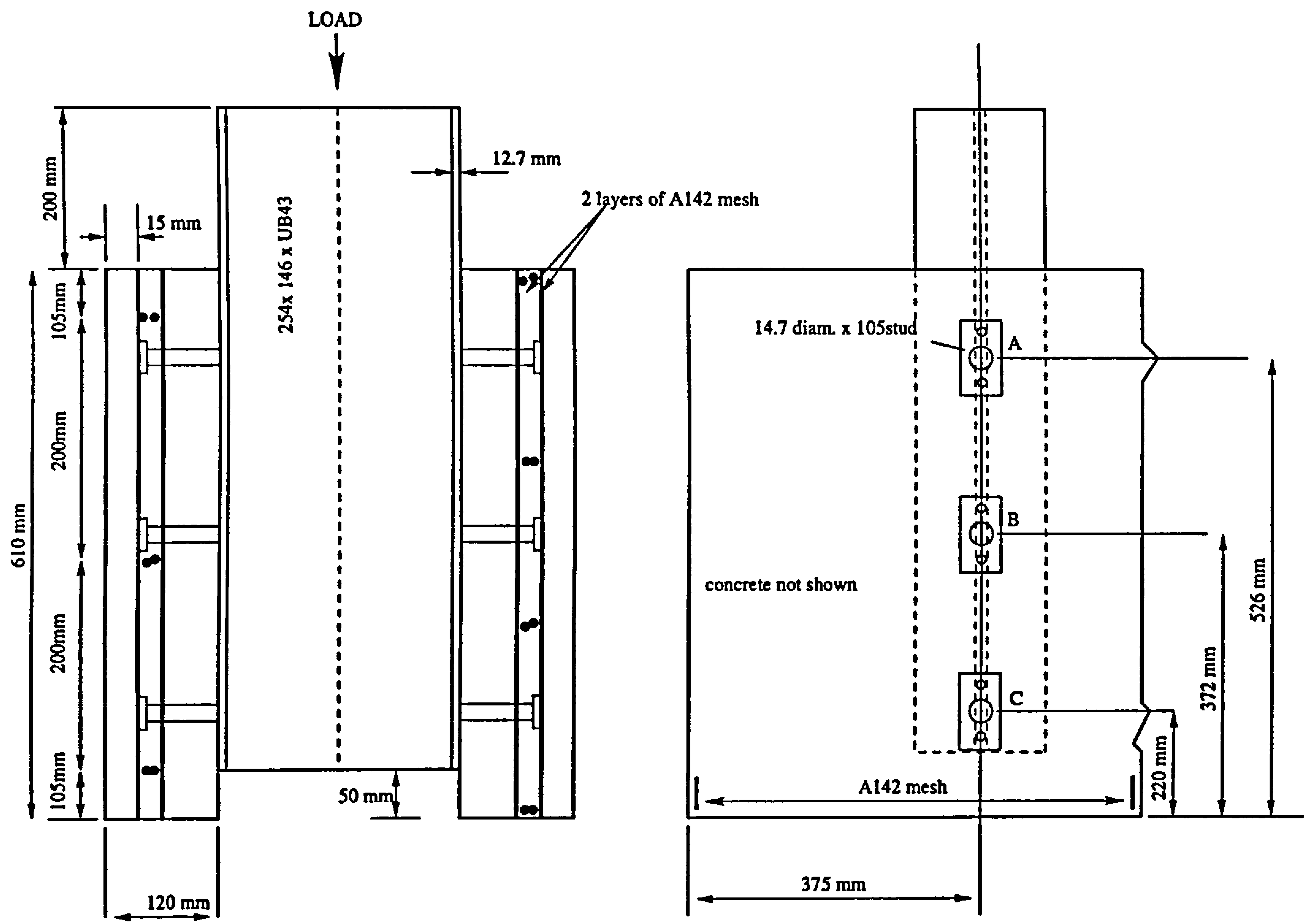


Figure 6.2 Test configuration for solid slabs S30-1

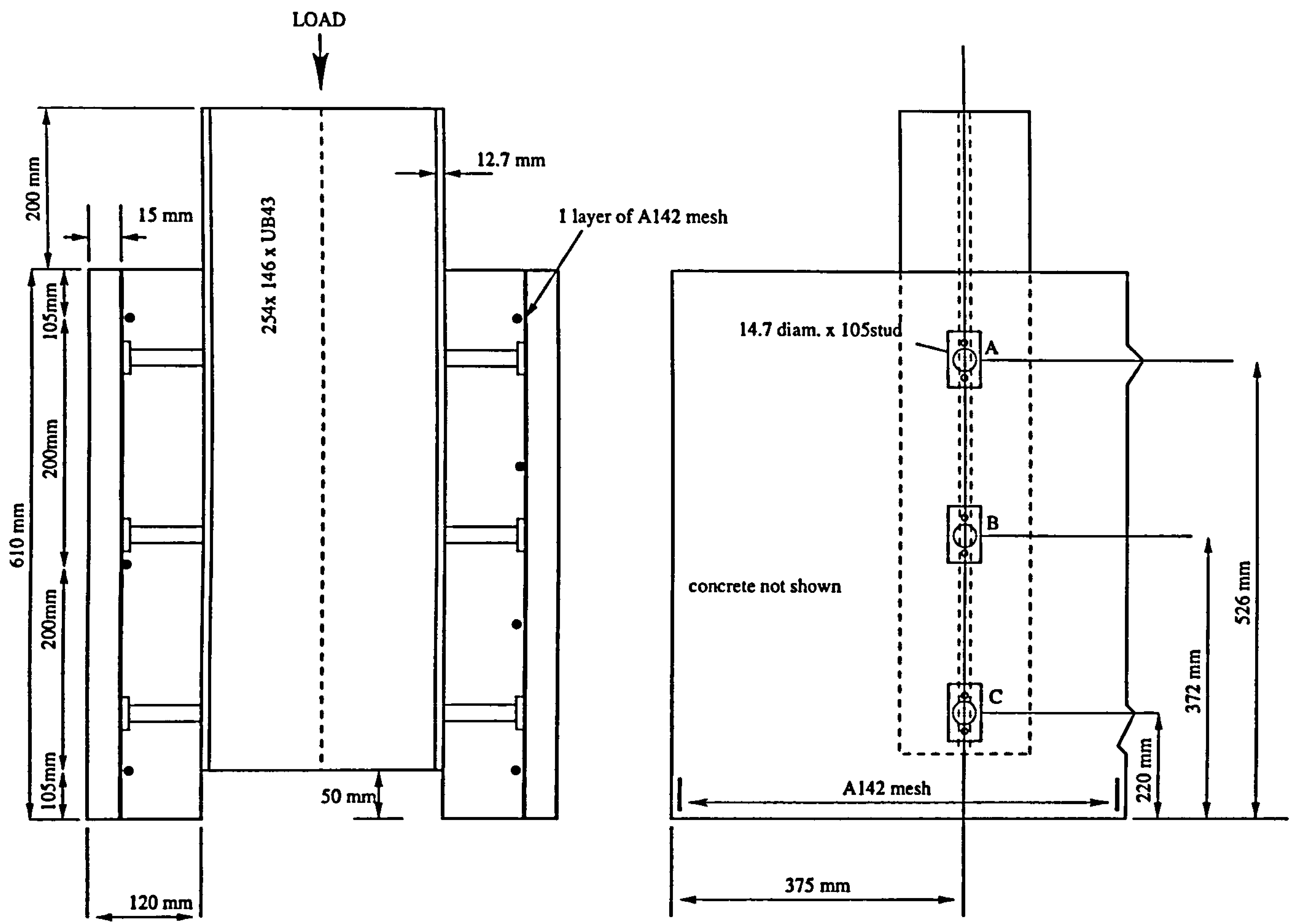


Figure 6.3 Test configuration for solid slab S30-2 and S30-3

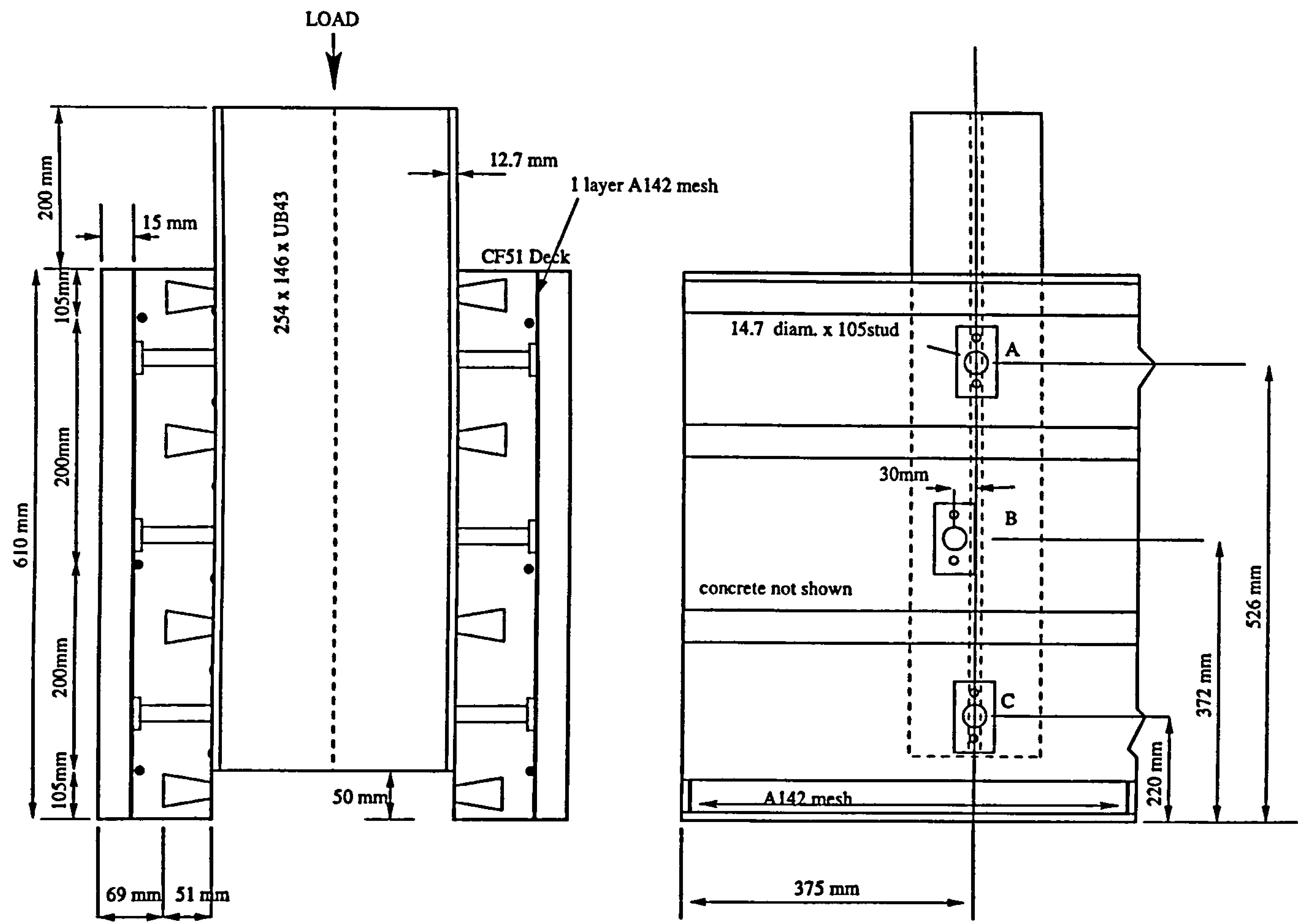


Figure 6.4 Test configuration using CF51 decking in specimen CF30-4

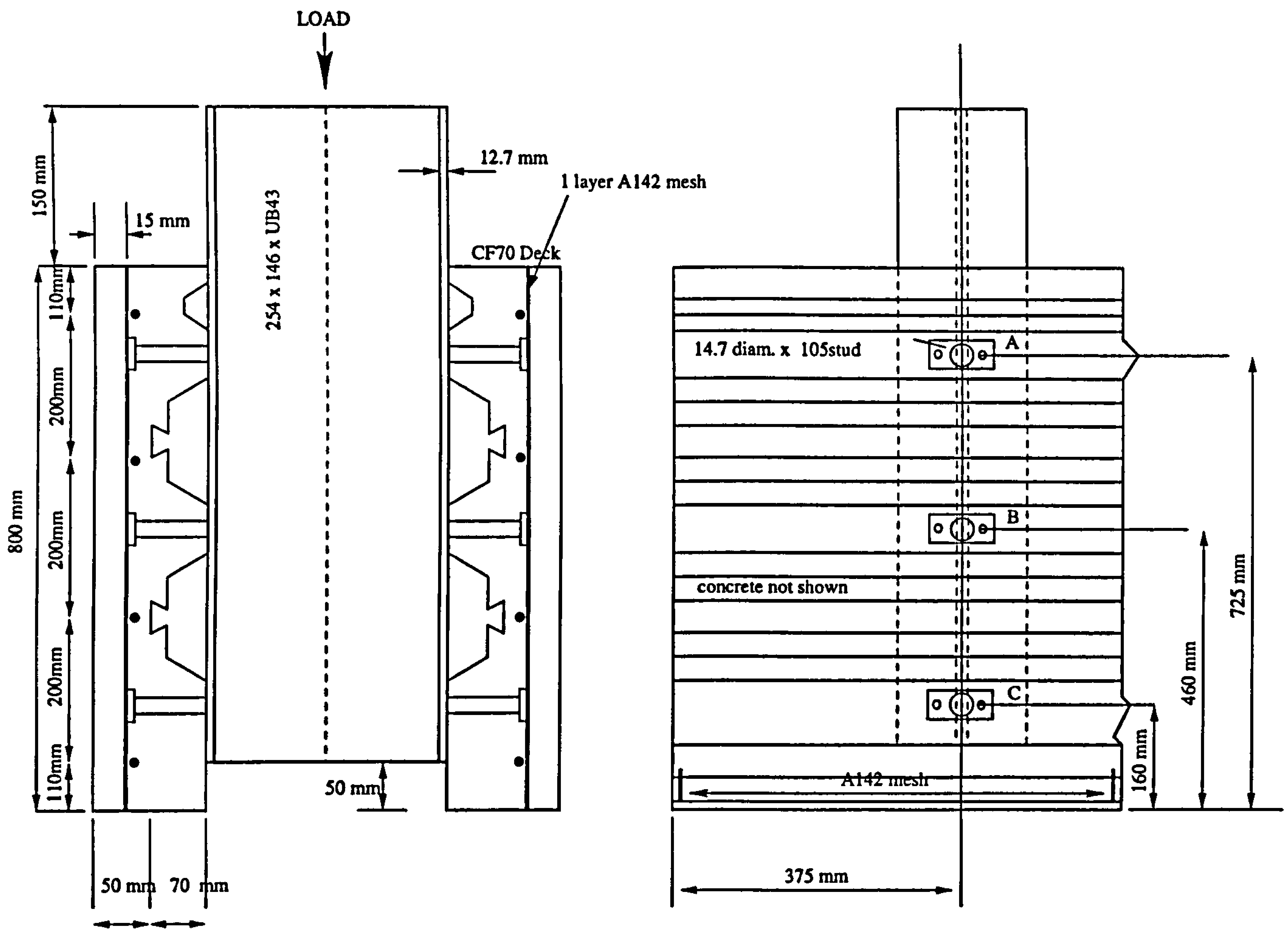


Figure 6.5 Test configuration using CF70 decking in specimen CF30-5

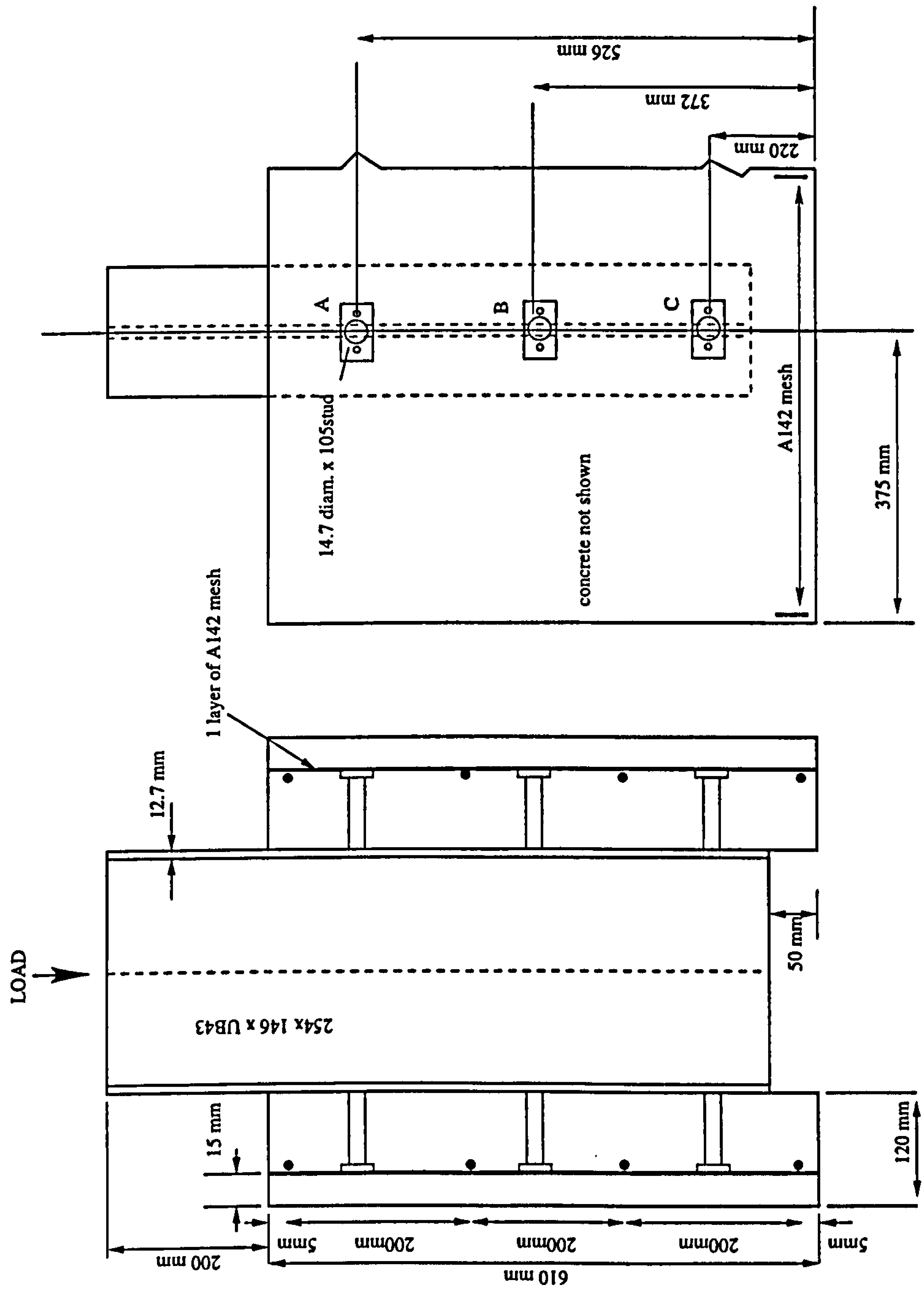


Figure 6.6 Test configuration for solid slab S30-6

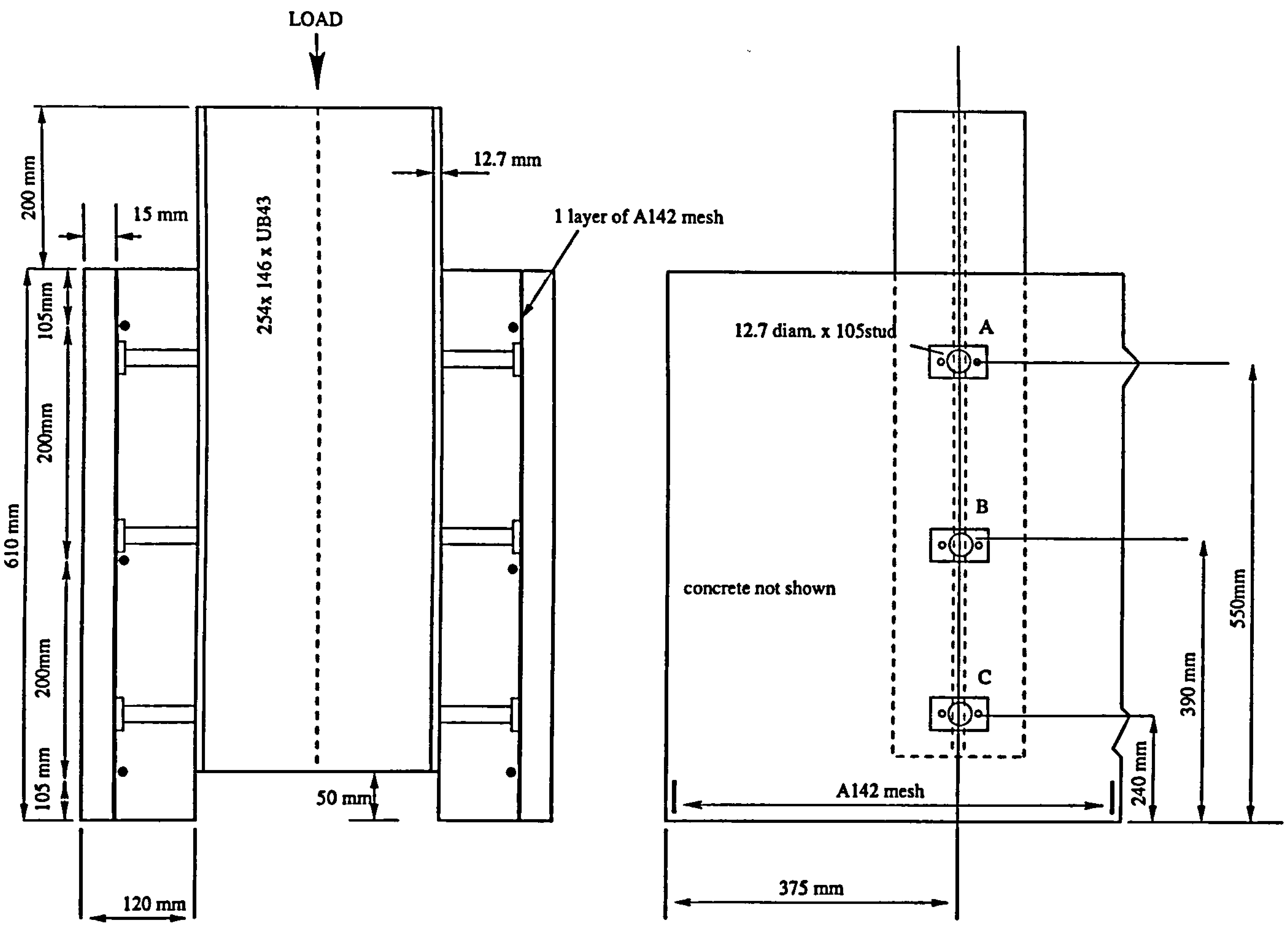


Figure 6.7 Test configuration for solid slab S30-7

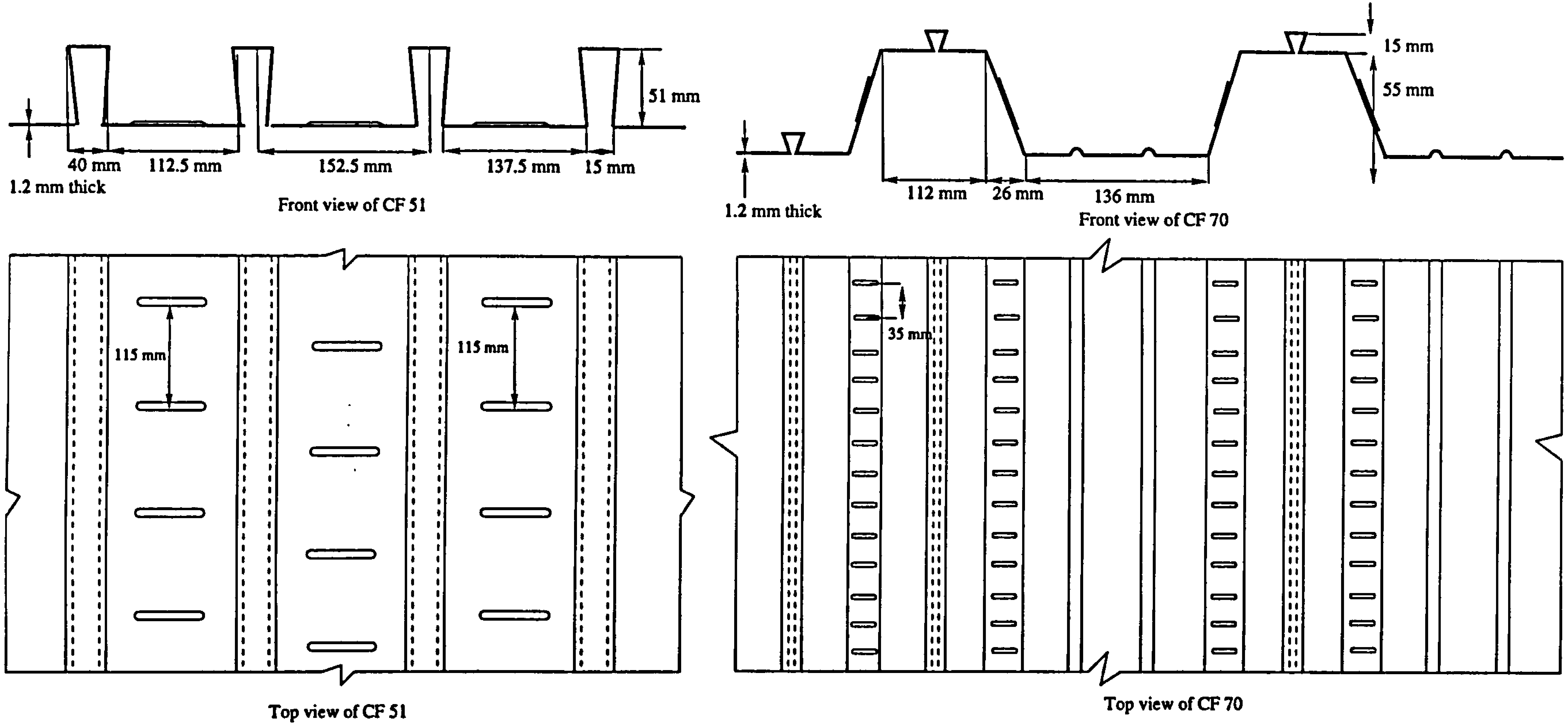


Fig. 6.8 : Geometry configuration of CF 51 and CF 70 metal decking

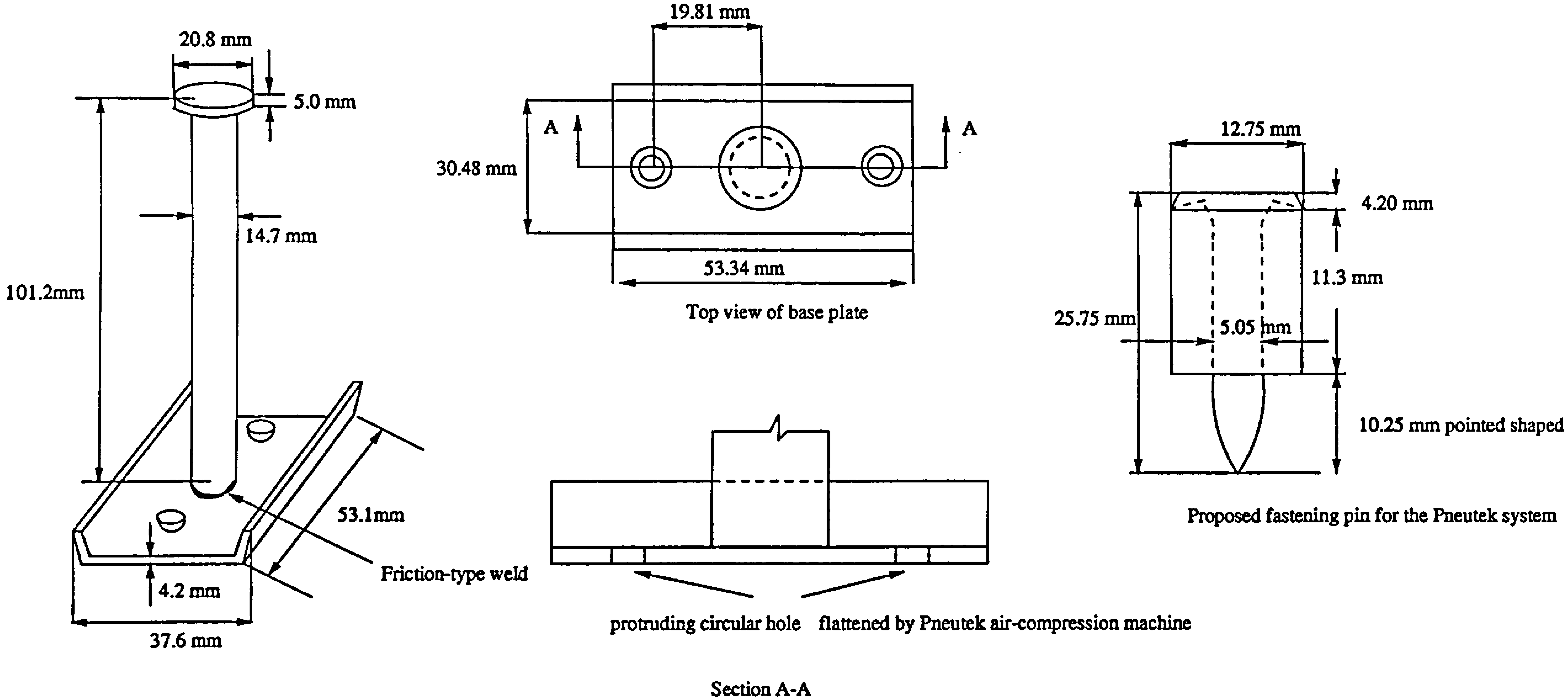


Figure 6.9: Geometrical characteristics of the Pneutek air-safe shear stud connector using the proposed pin

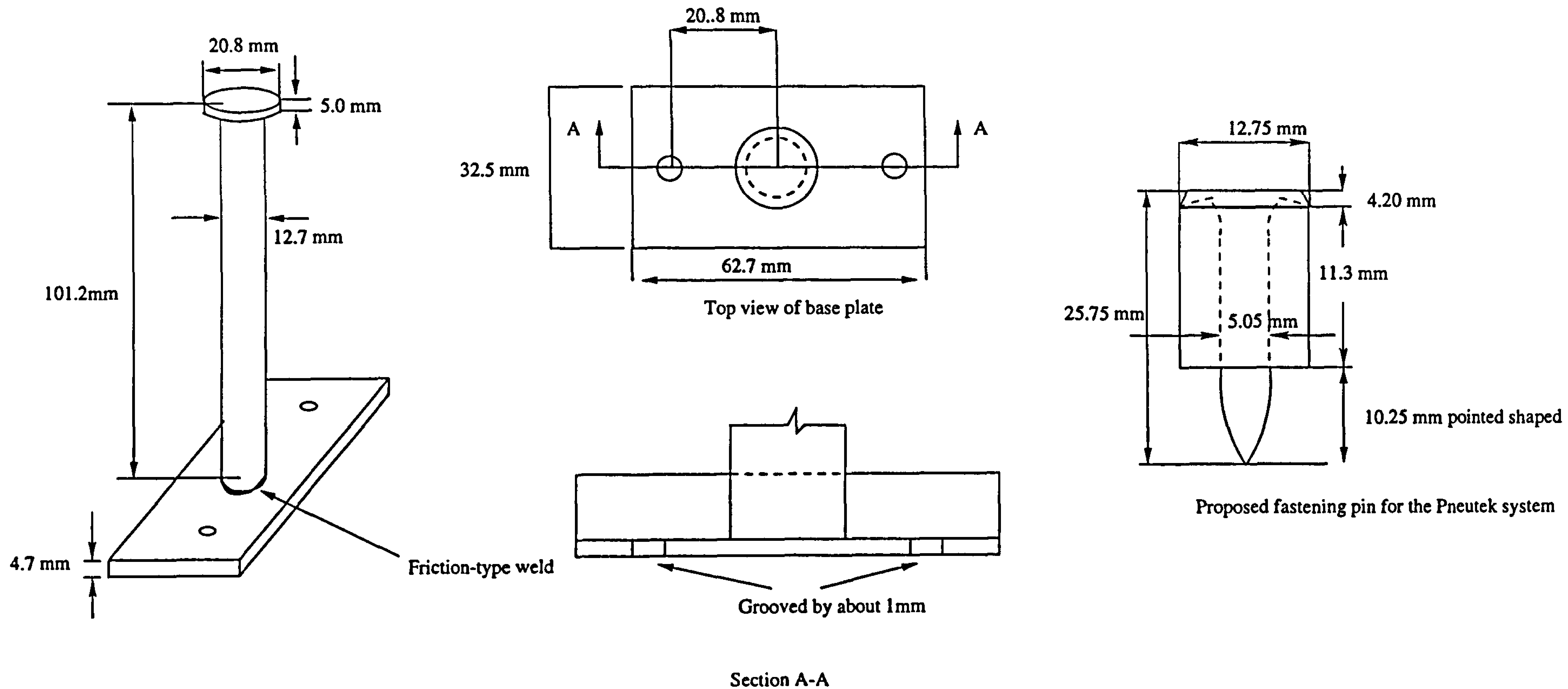


Figure 6.10: Geometrical characteristics of the Pneutek air-safe shear stud connector using the proposed pin

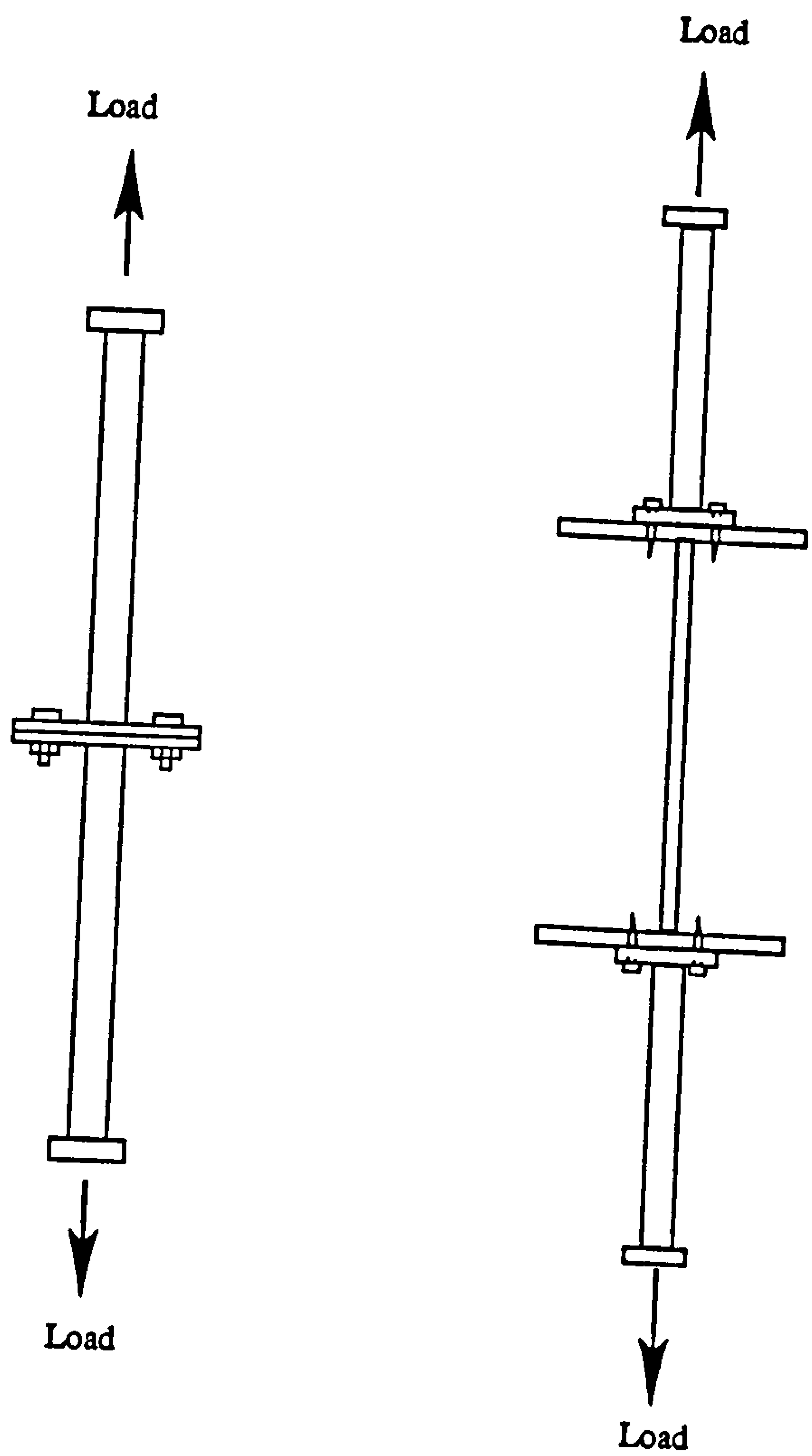


Fig 6.11. : Arrangements of studs tested for tensile strength

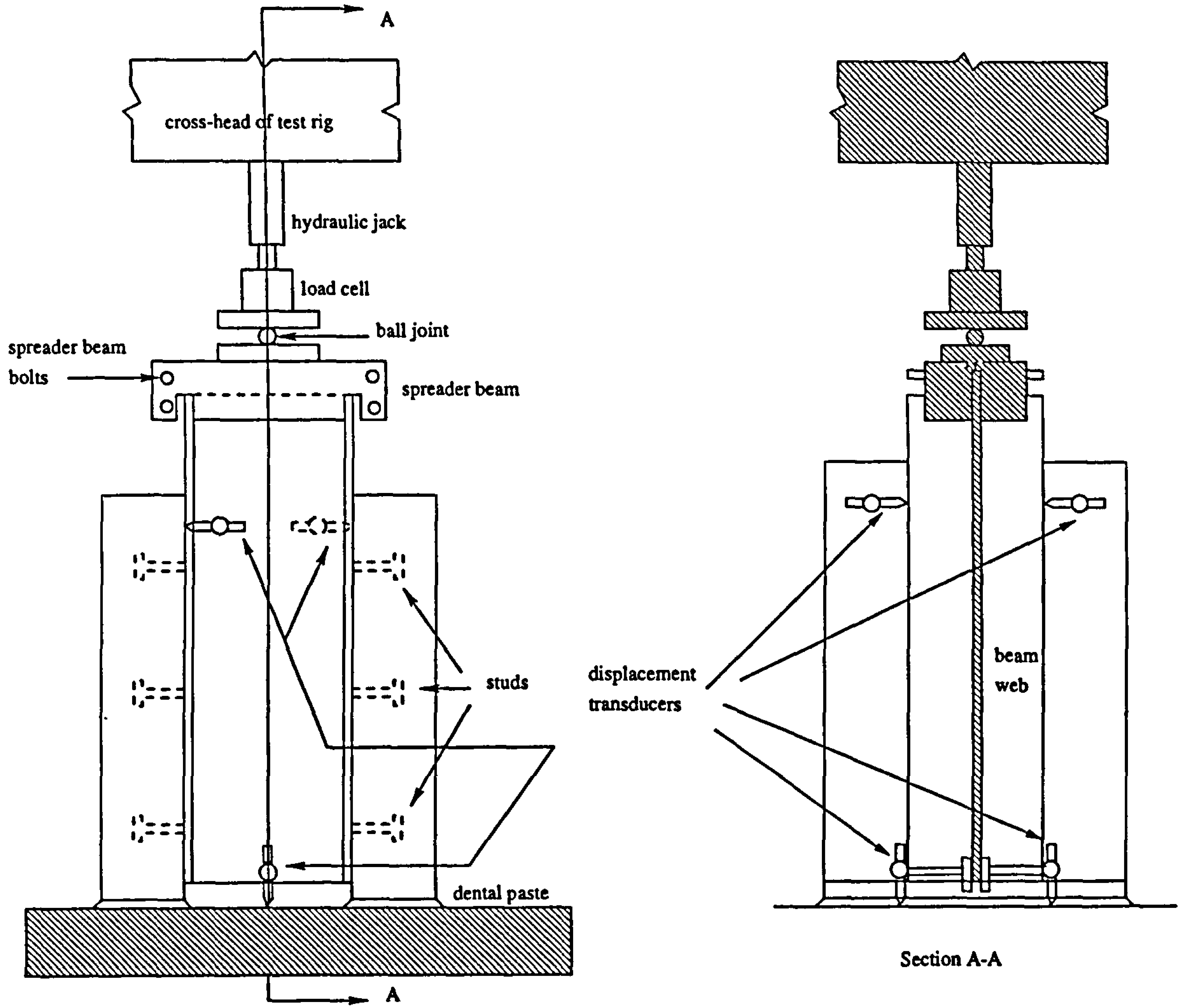


Fig 6.12 Test Rig

Fig. 6.13: Load–slip curve for Specimen S30–1

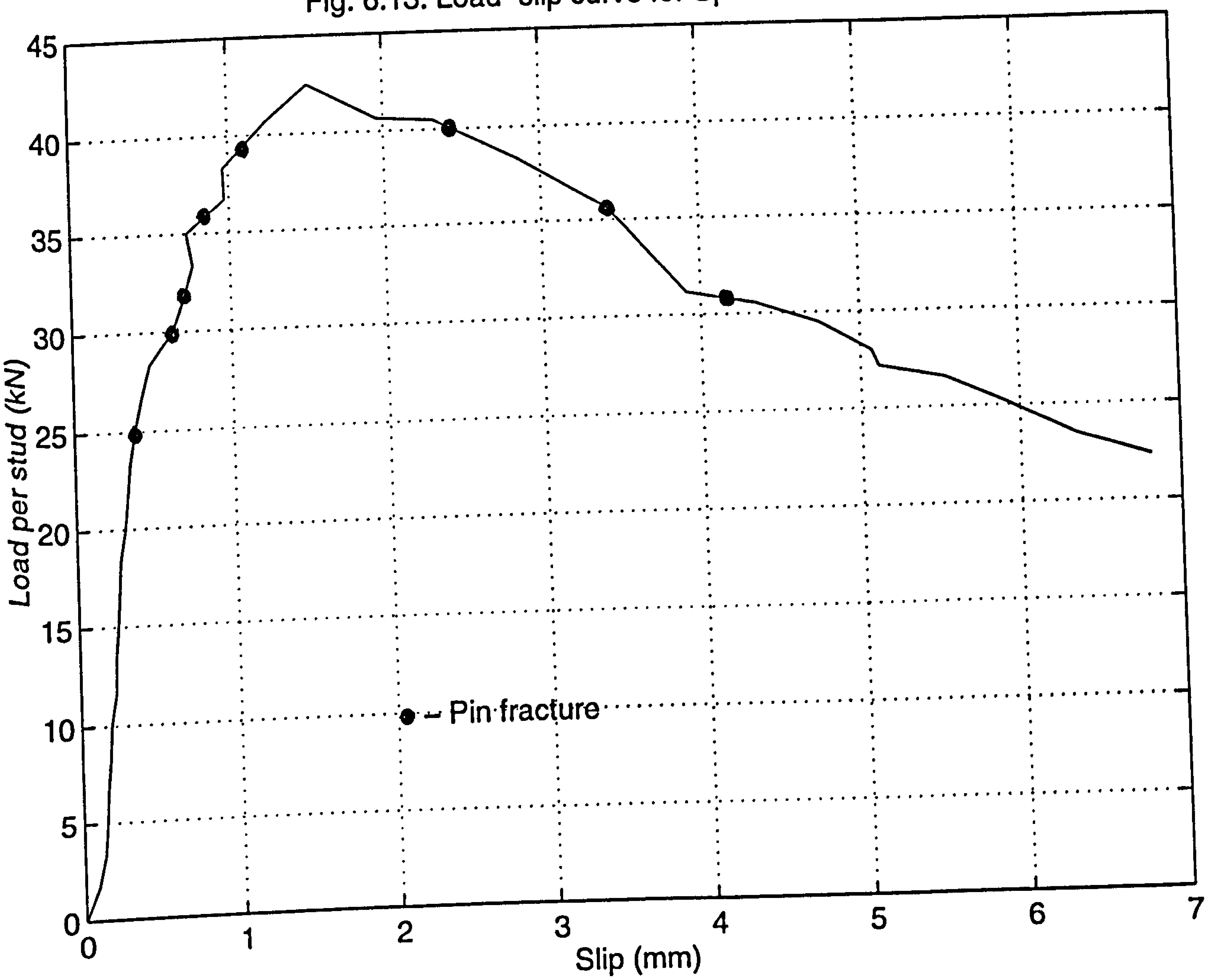
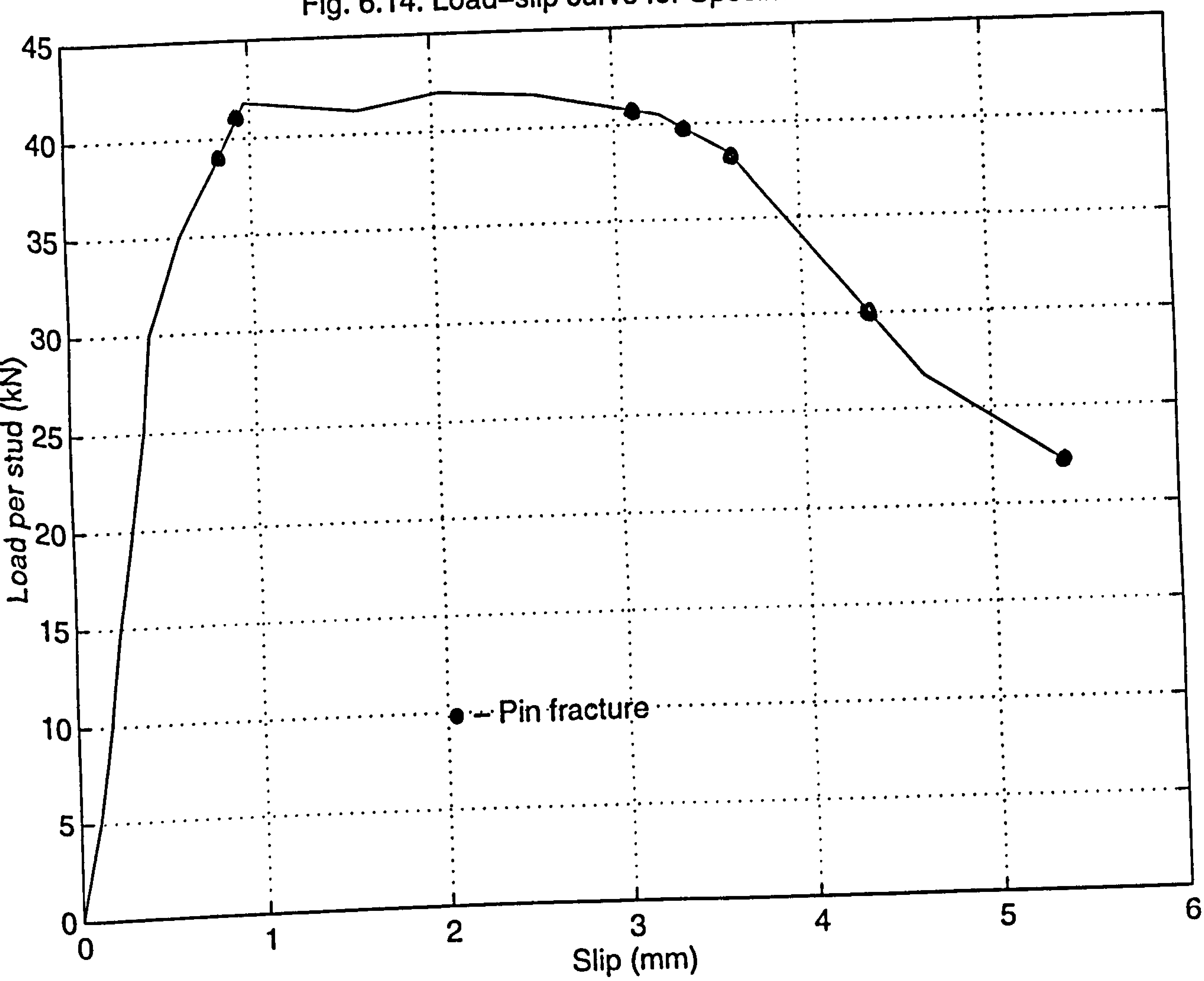


Fig. 6.14: Load–slip curve for Specimen S30–2



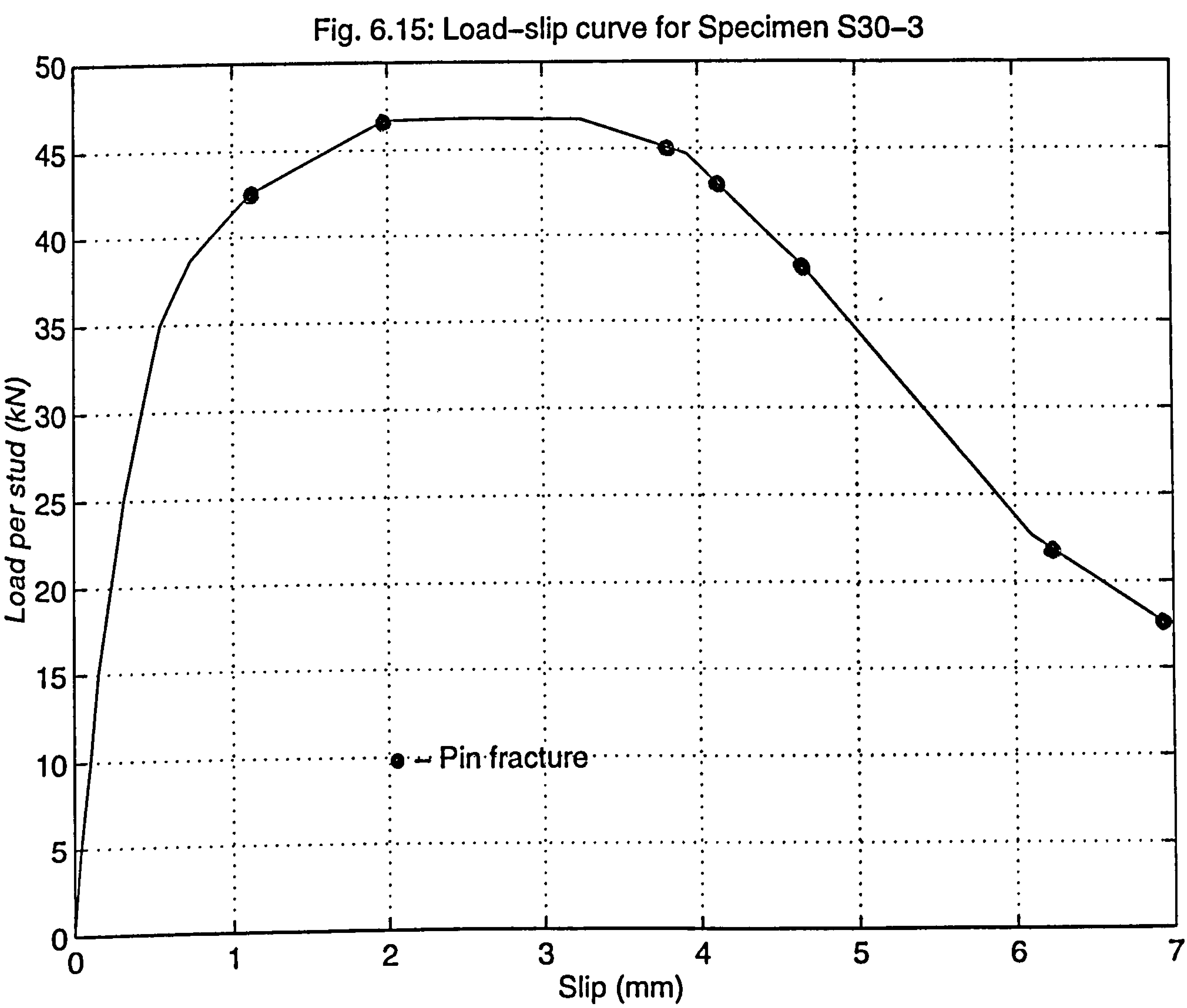


Fig. 6.15: Load–slip curve for Specimen S30–3
Load per stud (kN)
50
45
40
35
30
25
20
15
10
5
0
0
1
2
3
4
5
6
7
Slip (mm)
● – Pin fracture

Fig. 6.16: Load–slip curve for Specimen CF30–4

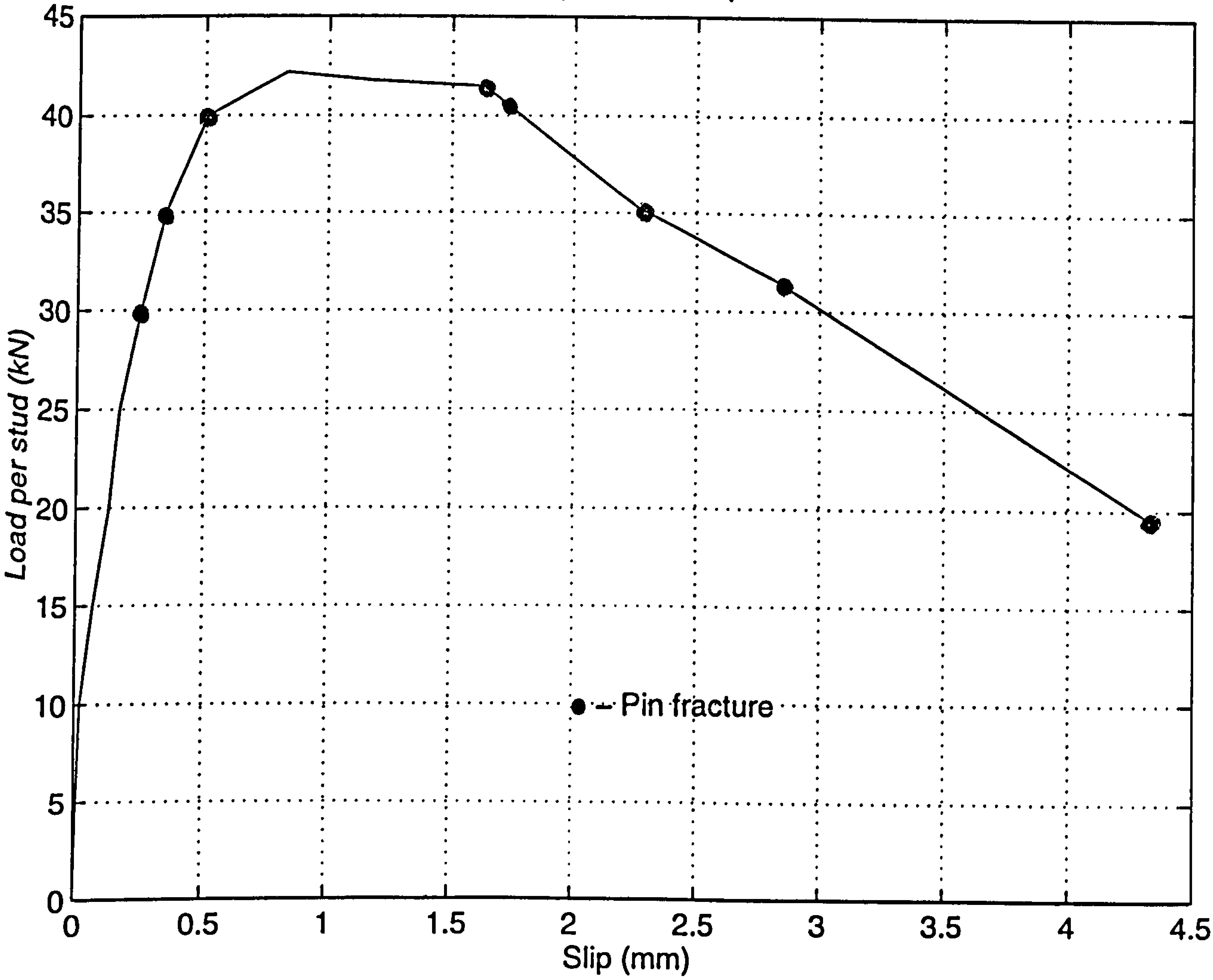


Fig. 6.17: Load–slip curve for Specimen CF30–5

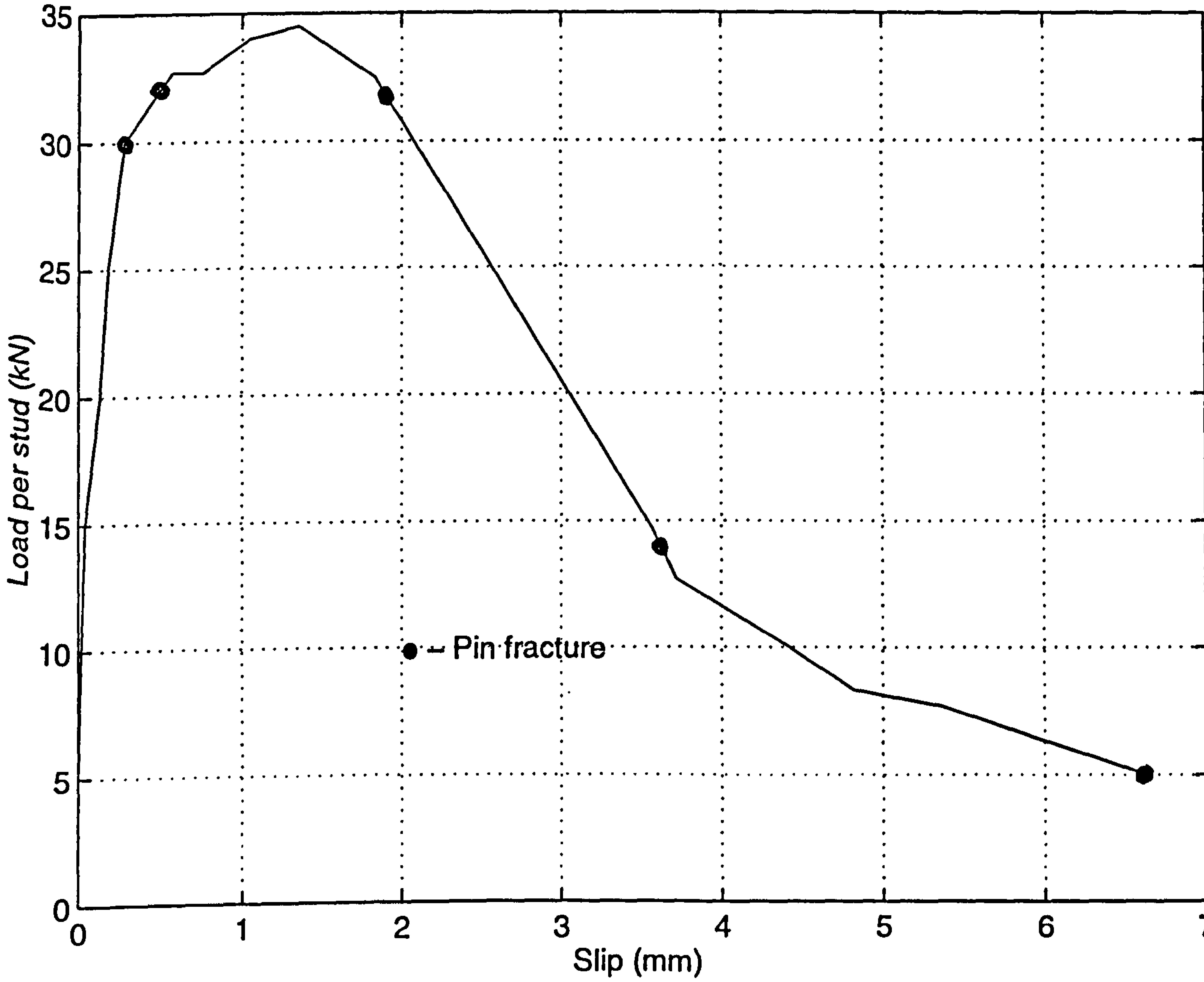


Fig. 6.18: Load-slip curve for Specimen S30-6

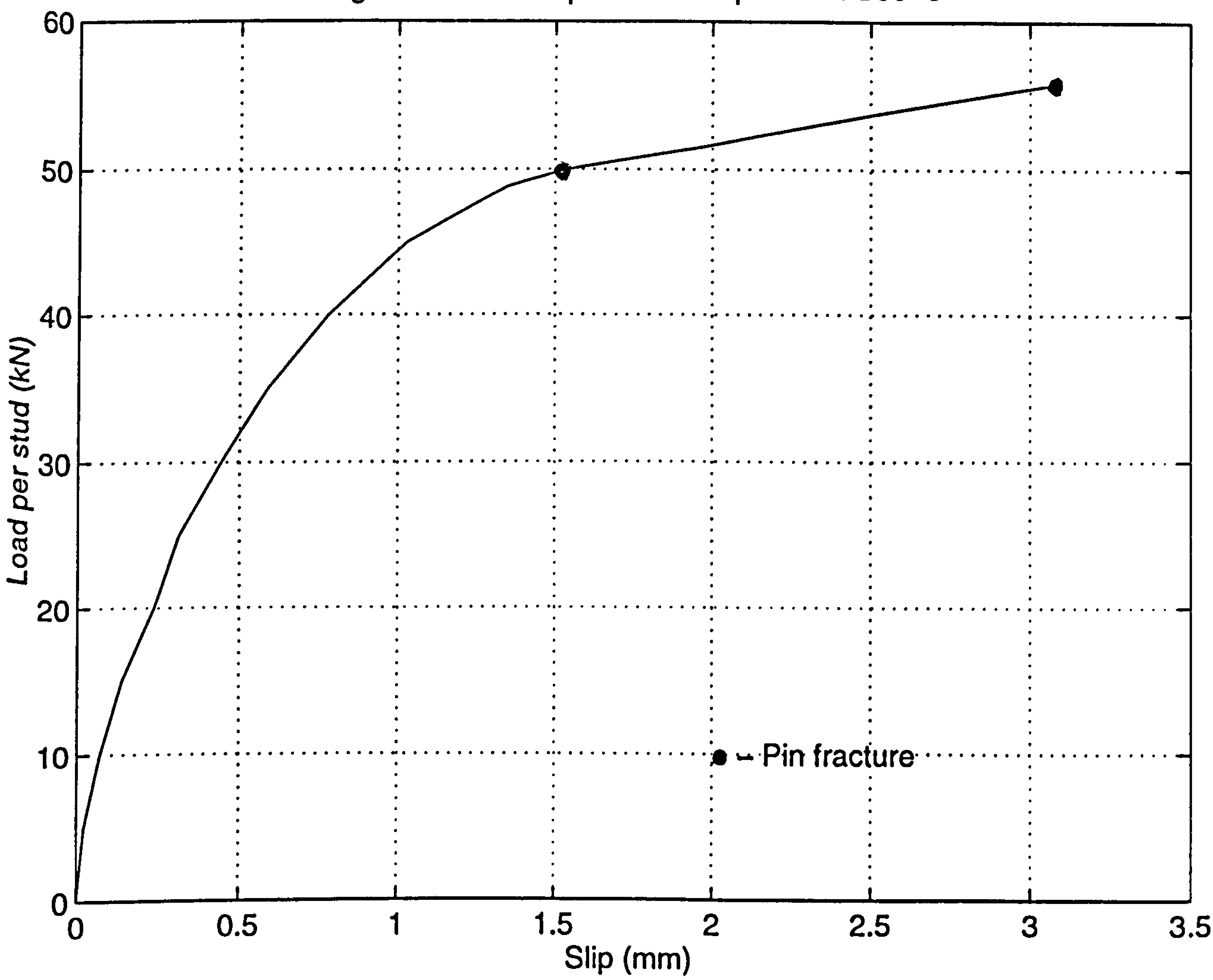
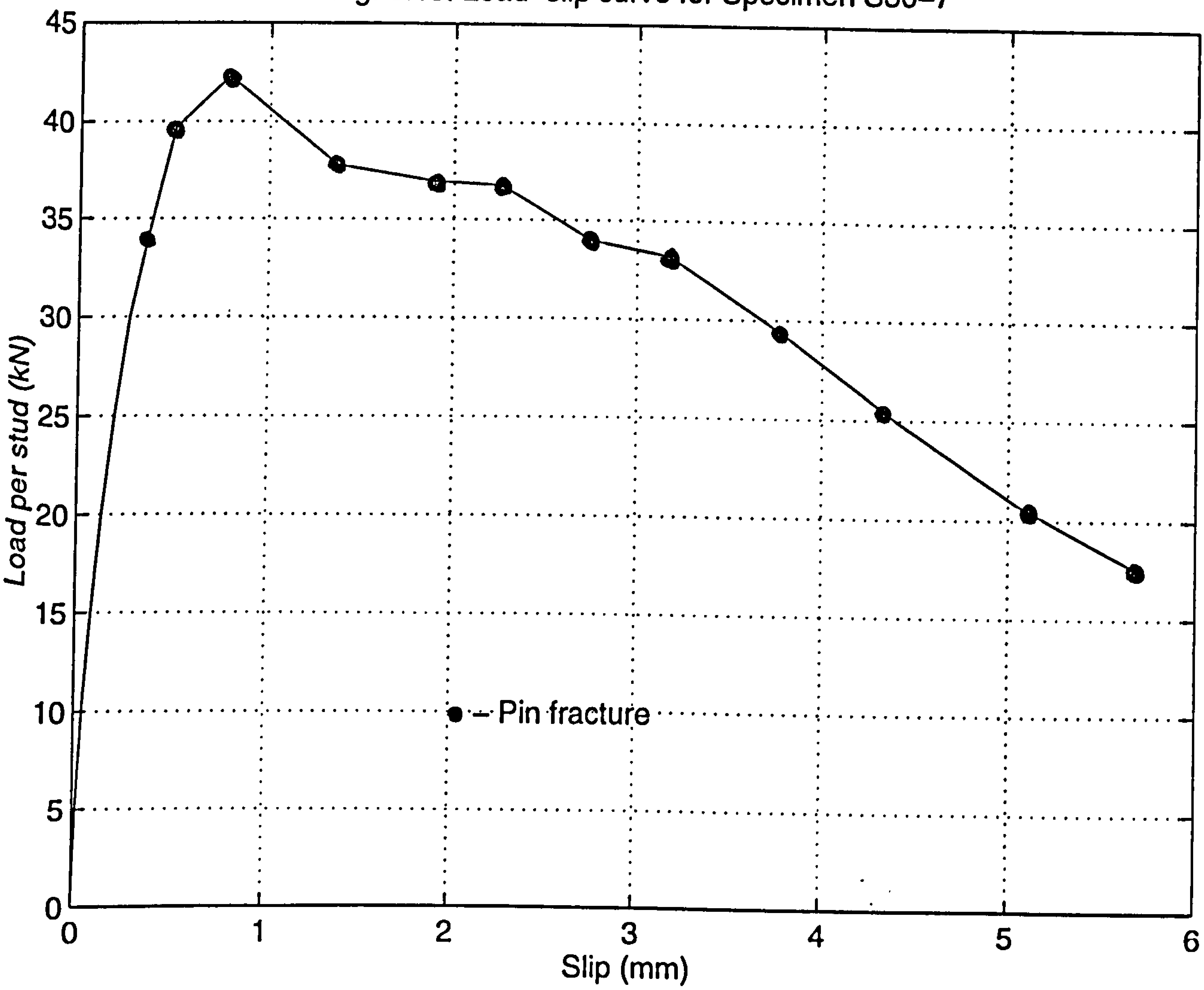
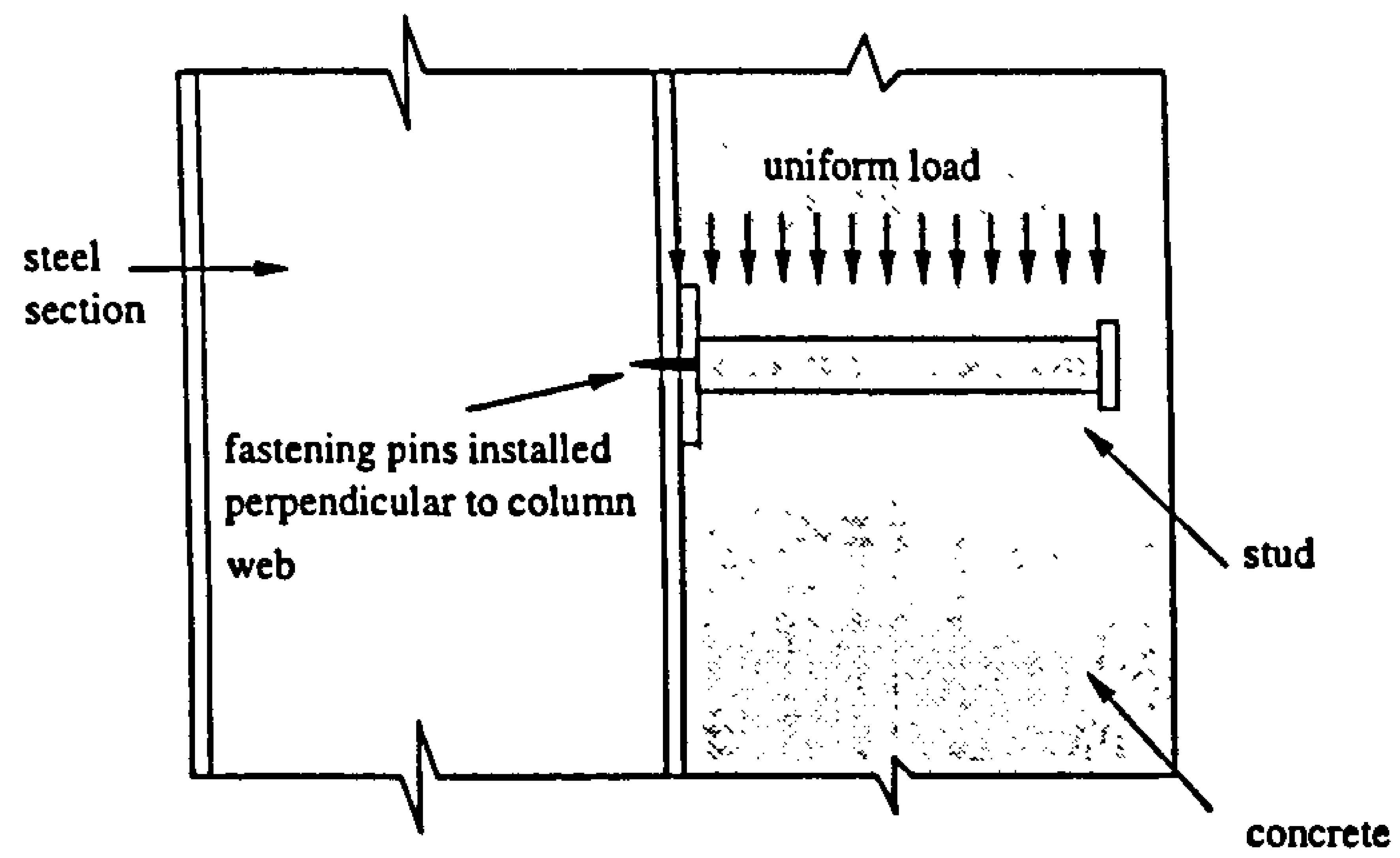
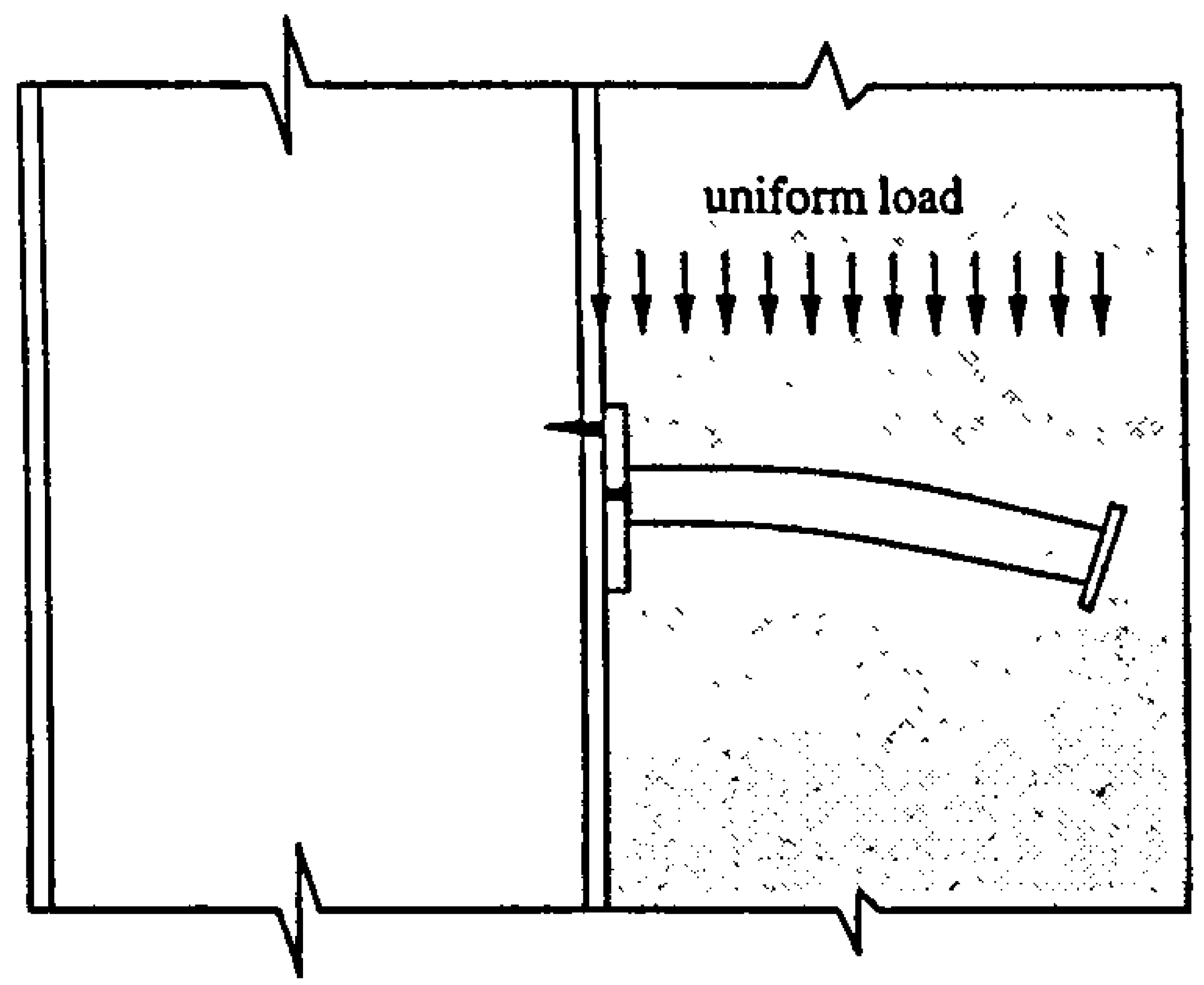


Fig. 6.19: Load–slip curve for Specimen S30–7

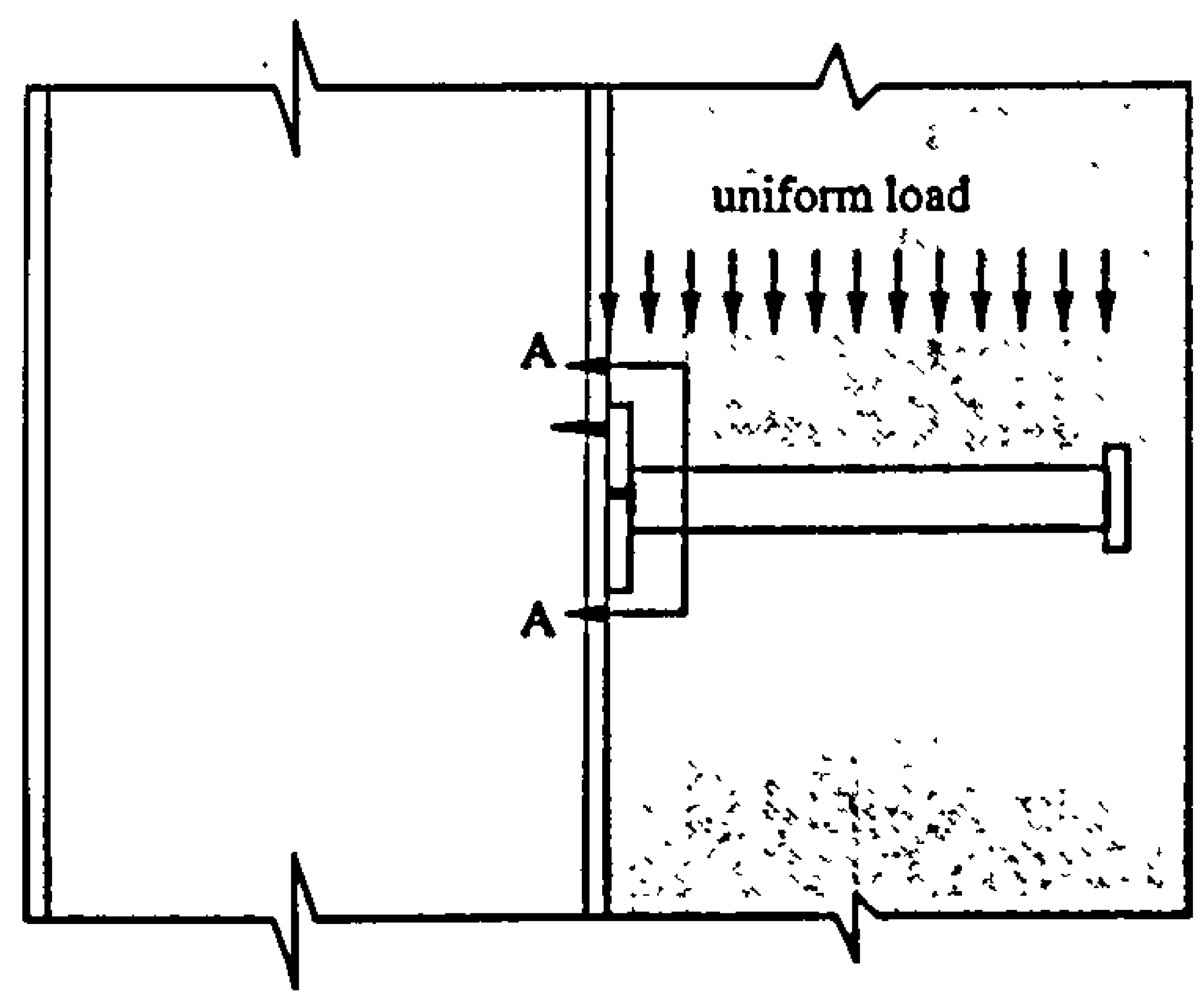




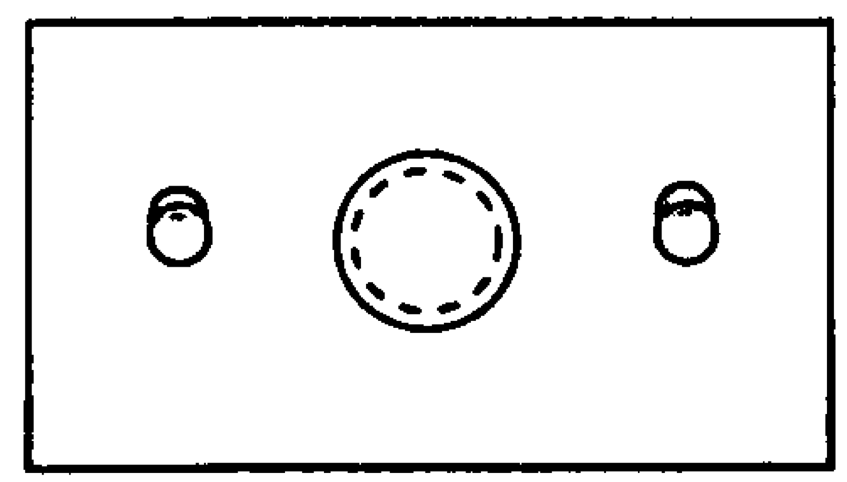
a) Uniform load contributed to shear and bending to fastening pins



b) expected failure of the stud system



c) failure of stud system from the tests



Section A-A

d) distortion of pins' holes due to shear and bending

Fig 6.20 Loading and failure modes on the stud syatem

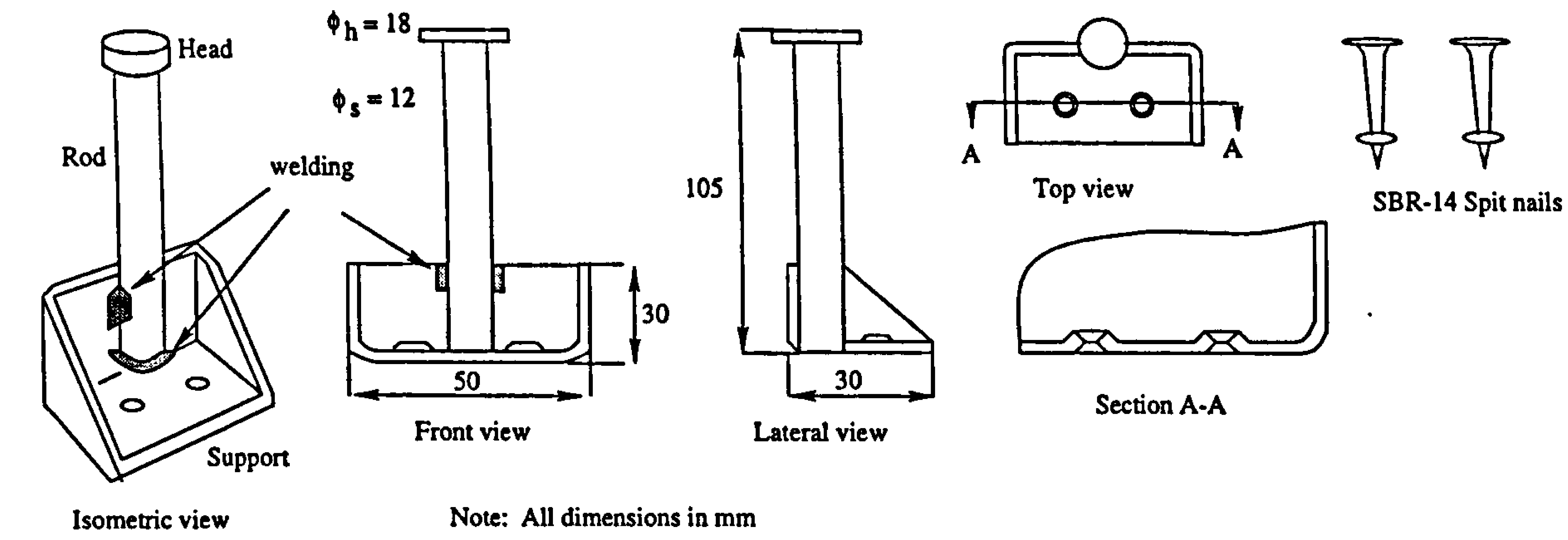


Fig. 6.21: Geometrical characteristics of the Spit shear connector.

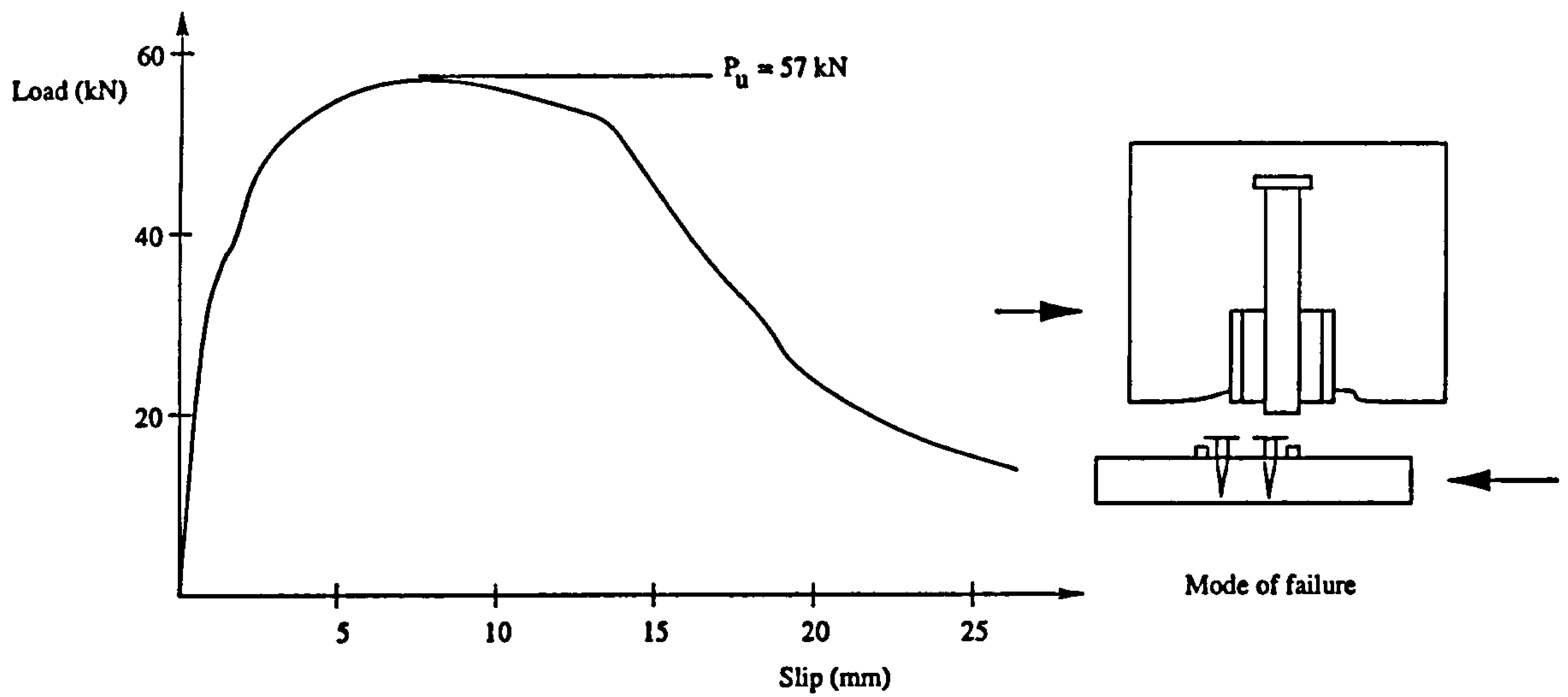


Fig. 6.22: Load- slip curve and mode of failure for the Spit connector.

Chapter 7

7.0 Conclusions and suggestions for further work

7.1 Conclusions

The research work presented in this thesis was carried out by the author from January 1994 to March 1997. The work described in Chapter 1 has contributed to a greater understanding of the theoretical and practical aspects of steel frame design for buildings. Detailed conclusions have been given at the end of each chapter. However, as a summary, final discussions and conclusions are given below.

A study on structural and economic aspects resulting from the use of semi-rigid connections in steel frames was carried out using computer software for plane frames. The detailing of the semi-rigid connections and the moment capacity are taken from standard tables[7.1] to ease the design of the frames. Two types of standard details were used; namely flush and extended end plate connections.

In braced frame design, the use of partial strength connections can contribute to savings in both the costs of construction and the weight of material. However, the savings in cost and those in material are not necessarily proportional but are quite closely related. The comparisons however are not conclusive as the study was only limited to the flush end plate connections for beams spanning 6m and only one fabricator was selected to determine the savings in cost. Further work is suggested later to improve the comparison on savings in both material and cost. The use of grade S275 steel contributed to a better percentage saving in material, with an average of about 7.8% compared with the use of grade S355 steel with an average of about 4.4%. However, the percentage saving~~s~~ for S355 calculated by the author did not take into account the effect of semi-rigid connections on deflection. The comparison between the two grades of steel is more realistic if this effect is taken into account as some of the beams designed in S355 steel were governed by deflection. The overall results however, proved that the use of partial-strength connections did contribute to a useful percentage saving which can be considered as a significant finding.

The study on unbraced frames bending about both column axes (Chapter 3) showed that it was not possible to design frames only by wind-moment analysis[7.2] to provide adequate resistance. However, with the use of Anderson and Islam's formulae[7.3] to limit ~~in~~ deflection, the stiffness and ultimate resistance can be improved. The advantage of the approach is that the frames can be analysed without the need of specialised computer software, but the problems of lack of sufficient stiffness in some frames detracts from the confidence with which designers apply the method to major axis frames. The connections used by the author, where the end plate is bolted to a thick plate which is then welded to column flanges, is not regarded as economical. Another assumption taken by the author was to consider the column base as fixed. However, in present British practice, this base is supposed to be taken as semi-rigid. To improve the analysis of the frames, a suggestion is also given in 7.2.

The study on frames with composite beams (Chpater 5) showed that the design of composite beams in unbraced frames is dependent on the wind and gravity loading applied. The composite beams were not always safe and economical when only vertical load was taken into account in their design. The moment capacity of the connections and the performance of the frames accounting for cracking should also be considered. The crack distance developed in the analysis determined in Chapter 5 was found to be very small and insignificant to the performance of the frame. The results in Chapter 5 proved that frames with low wind load were more suitable to be designed with composite beams where the moment capacity of the connection was determined from the bare steel connection. The use of composite joints is suggested for future work as described in 7.2. This may further improve the performance of the frames.

The study on push tests for a non-conventional stud system was carried out to understand more fully its strength and ductility. The results described in Chapter 6 showed that more improvements need to be made to the stud system, to increase its maximum resistance and ductility. The results of the experiments showed that the strength and the ductility were not achieved as expected. The remedy to improve the performance is discussed later in this chapter.

7.2 Suggestions for further work

To further improve the research described in this thesis, suggestions for further work are listed as follows:

1. For braced frames, further study should be done on longer beam spans such as those of 12 metre. Optimum design, to balance the effect of the increase in column size due to connection moment against the reduction in beams should also be studied, using various types of joints. The contribution of stiffness from semi-rigid joints to reduce the deflection of beams needs to be considered. Other future work in this area is to design the beam as composite and compare it with bare steel design.
2. Further studies are suggested for unbraced frames with column base assumed partially fixed so as to investigate the possible deterioration in the stability of the frame. However, this suggestion will increase the deflection of the frame. Therefore, further studies on this area should also include the frame partially stiffened by the existing structural components within the building such as cladding, lift cores, and staircases. Study on proposed minor-axis connections used should take into account the cost and the difficulty in construction. To improve the fabrication cost and to ease the difficulty in construction, the end plate of the connection should be welded directly to the column flanges without the need of thick plate. However, more experimental tests are needed to understand the performance of this type of connection. Other future studies can also be undertaken on a comparison between partially braced and unbraced frames with semi-rigid connections to determine the percentage saving.

3. For minor axis joints, the actual moment capacity and stiffness of the standard wind moment connections, connected to the column web should be examined experimentally in full scale tests. The experimental tests should also be used to check that end plates bolted and welded to the web of the column do not sacrifice the ductility of the connections. The results can then be compared with theoretical values which may be established from three dimensional finite element analysis by modelling the joints to be the same as the actual tests.
4. For steel frames designed with composite beams, a further study is needed to determine the limiting wind speed for the design of such frames if increase in section is to be avoided. Further work should be undertaken on frames with greater numbers of bays and longer beam spans. To improve the moment capacity of connections, further studies are needed on connections considered as composite connections. The contribution to the moment capacity in composite joints will be dependent on whether sagging or hogging moment is developed at the joints.
5. For the stud system, more experimental tests are needed to establish its performance. To improve this system, the geometrical configurations of the base plate should be modified by taking into account the stress distribution around the fastening pins. The latest information given from Pneutek[7.7] has revealed that stronger pins are available for future study.

References.

[7.1] Joint in Steel Construction: Moment Connections, Steel Construction Institute, Ascot, 1995.

[7.2] Anderson, D., Reading. S.J., and Kavianpour, K., "Wind-moment design for unbraced frames", The Steel Construction Institute Publication No. 082, 1991.

[7.3] Anderson, D., & Islam, M.A., "Design of multi-storey frames to sway deflection limitations", The Structural Engineer, Vol 57B, March 1979, pp11-17.

[7.4] Gomes, F.C.T. - "Etat limite ultime de la l'ame d'une colonne dans un assemblage semi-rigide d'axe faible", Internal Report No. 203, MSM, University of Leige, 1990.

[7.5] Eurocode 3, ENV - 1993-1-1, Revised Annex J, Design of Steel Structures, CEN, European Commitee for Standardization, Document CEN/TC 250/SC 3-N419 E, Brussels, June 1994.

[7.6] I-DEAS, Users Manual, Department of Engineering, University of Warwick, United Kingdom, 1995.

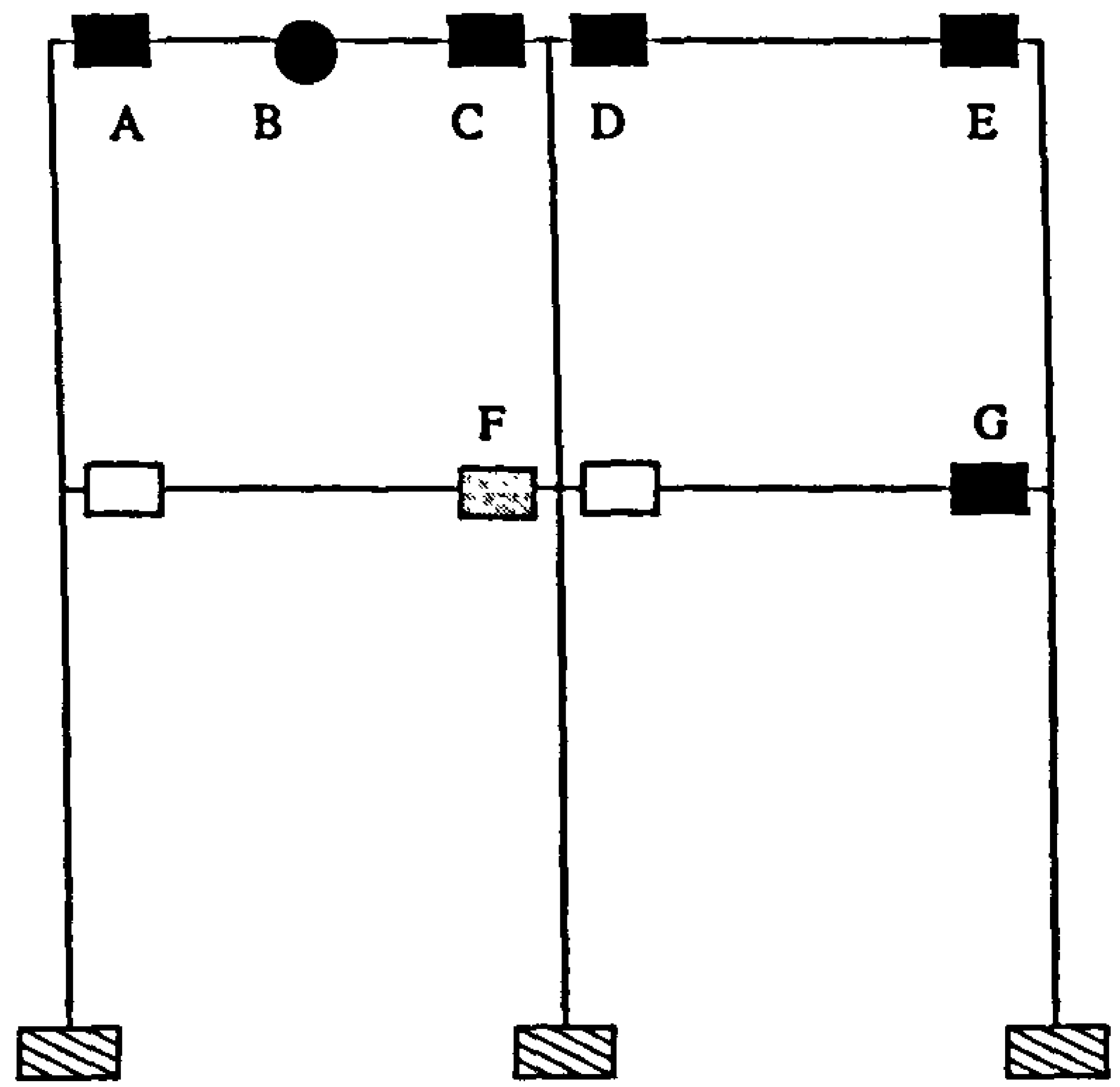
[7.7] Pneutek, Inc., Engineering Department, 29 Flagstone Dr., Hudson, NH 03051-9970, U.S.A.

Appendix A

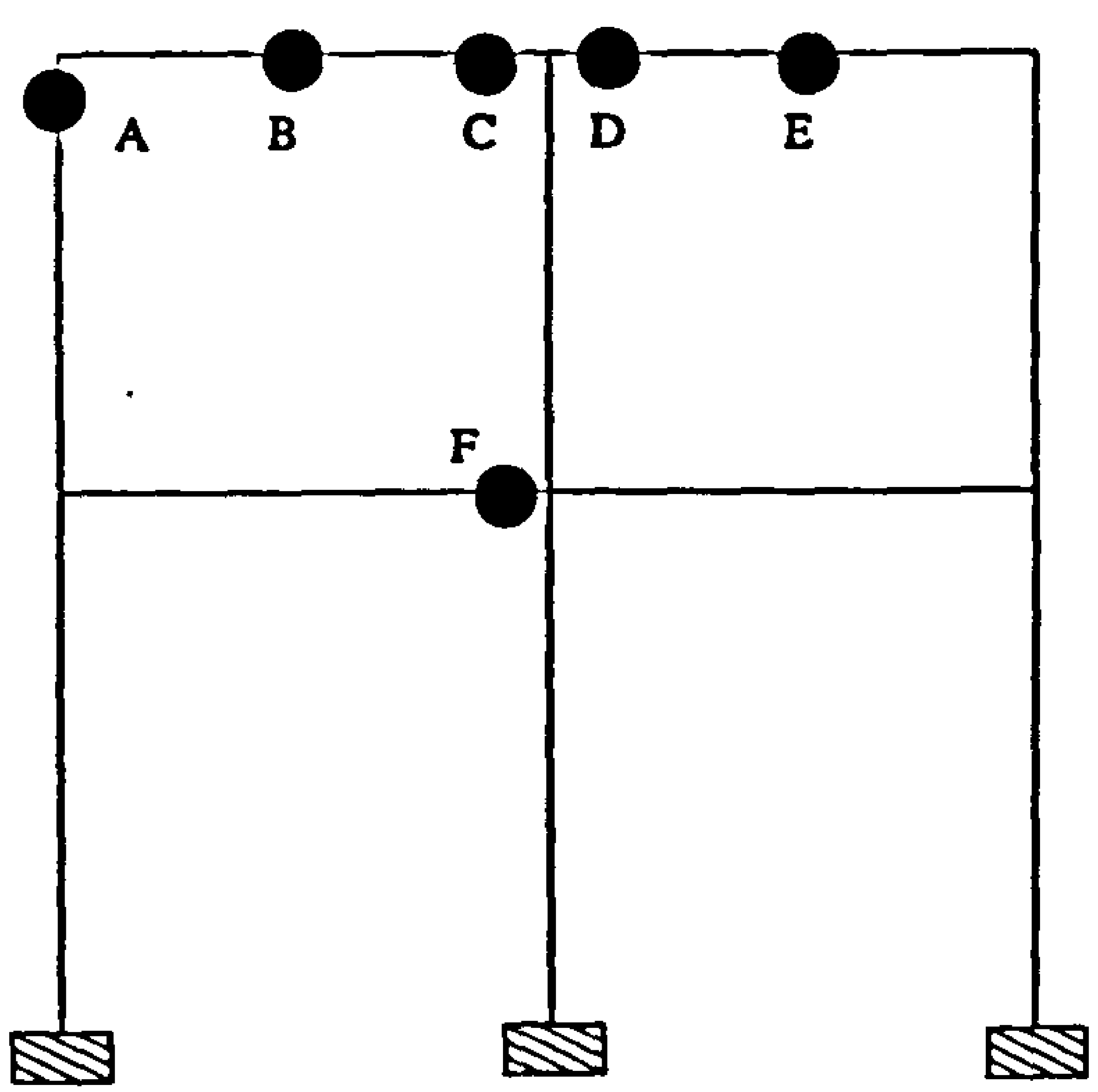
Plastic-hinges in minor-axis “wind-moment” frames.

Plastic-hinges in minor -axis "wind-moment" frames.

Semi-Rigid Frame (Section Designation II)



Rigid Frame (Section Designation II)



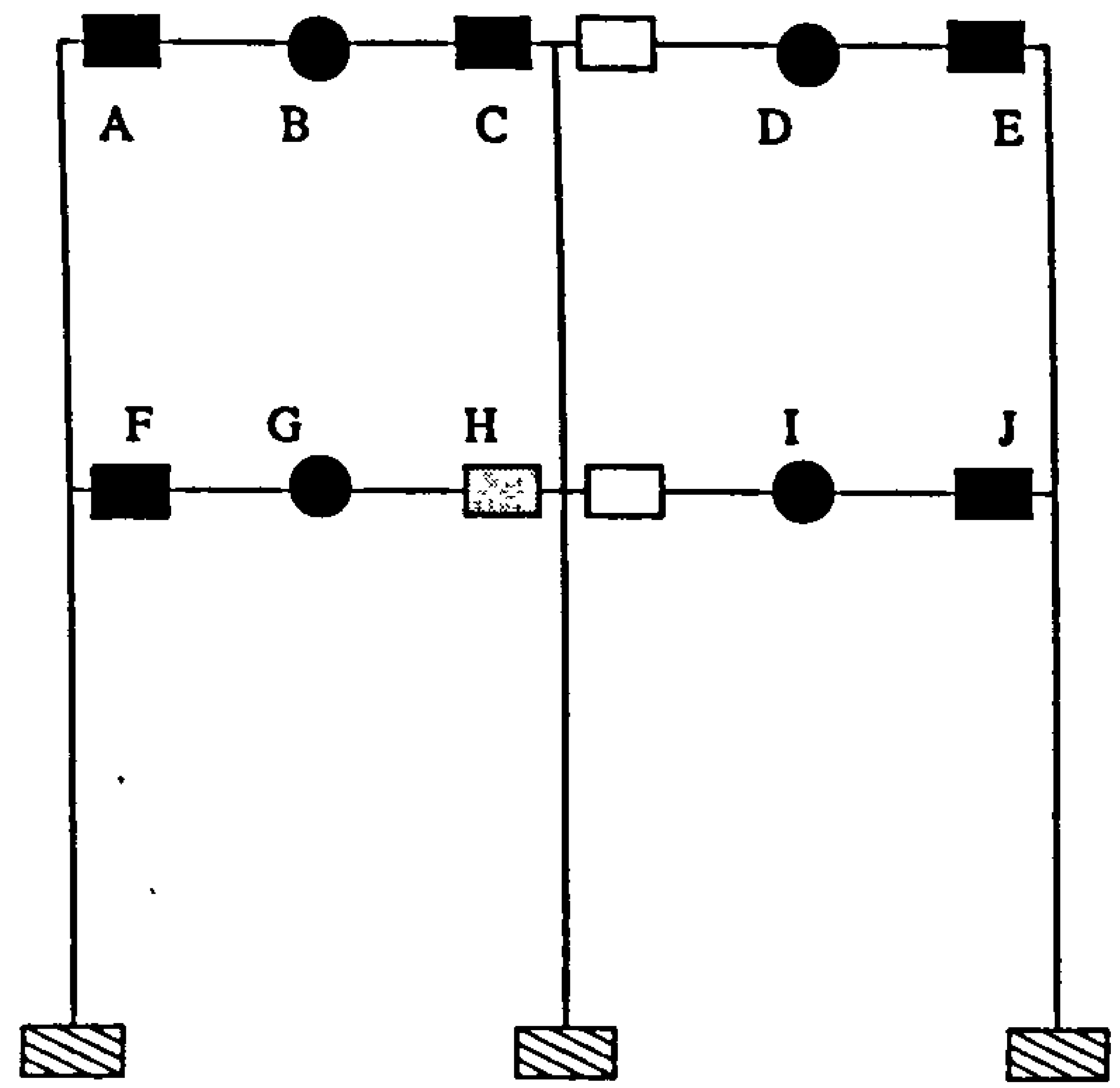
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.31	1.83
B	1.26	1.68
C	1.26	1.57
D	1.26	1.78
E	1.31	1.70
F	< 1.00	N/A
G	1.31	N/A

Key:

- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

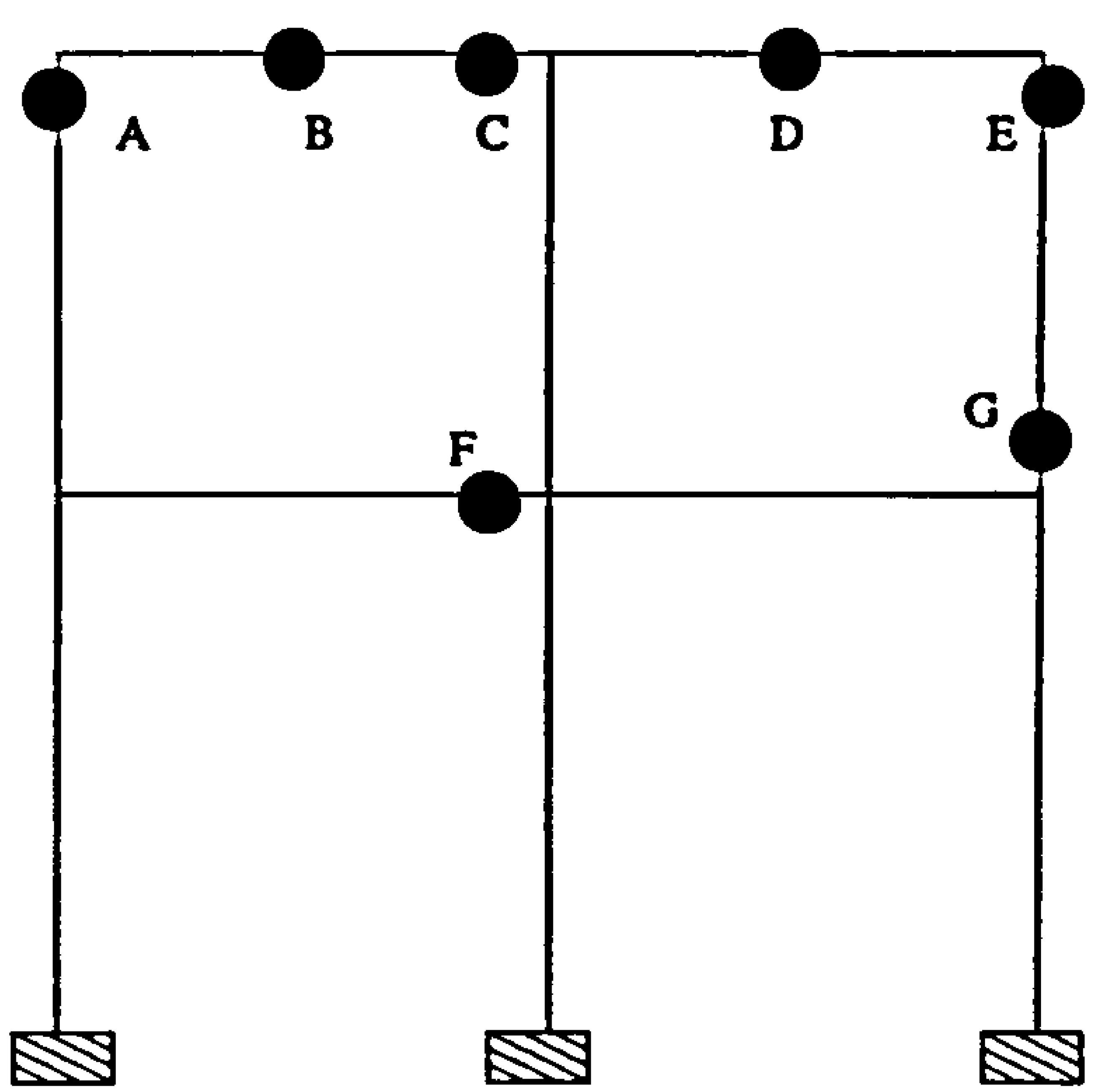
Frame 1
Load Case 1

Semi-Rigid Frame (Section Designation II)



Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.59	2.21
B	1.49	2.01
C	1.49	1.82
D	1.53	2.05
E	1.53	2.18
F	1.59	2.01
G	1.59	2.20
H	< 1.00	N/A
I	1.59	N/A
J	1.53	N/A

Rigid Frame (Section Designation II)

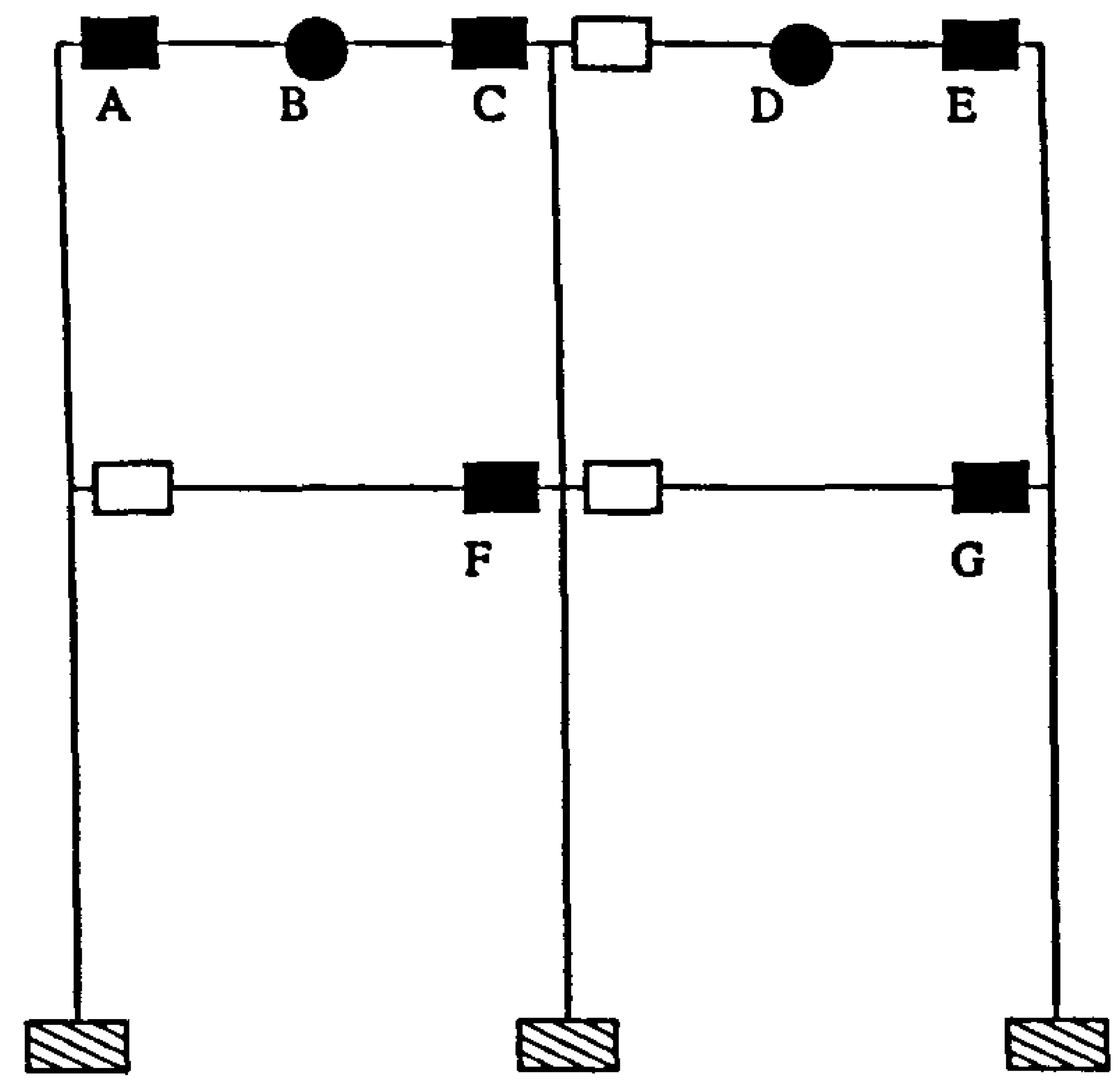


Key:

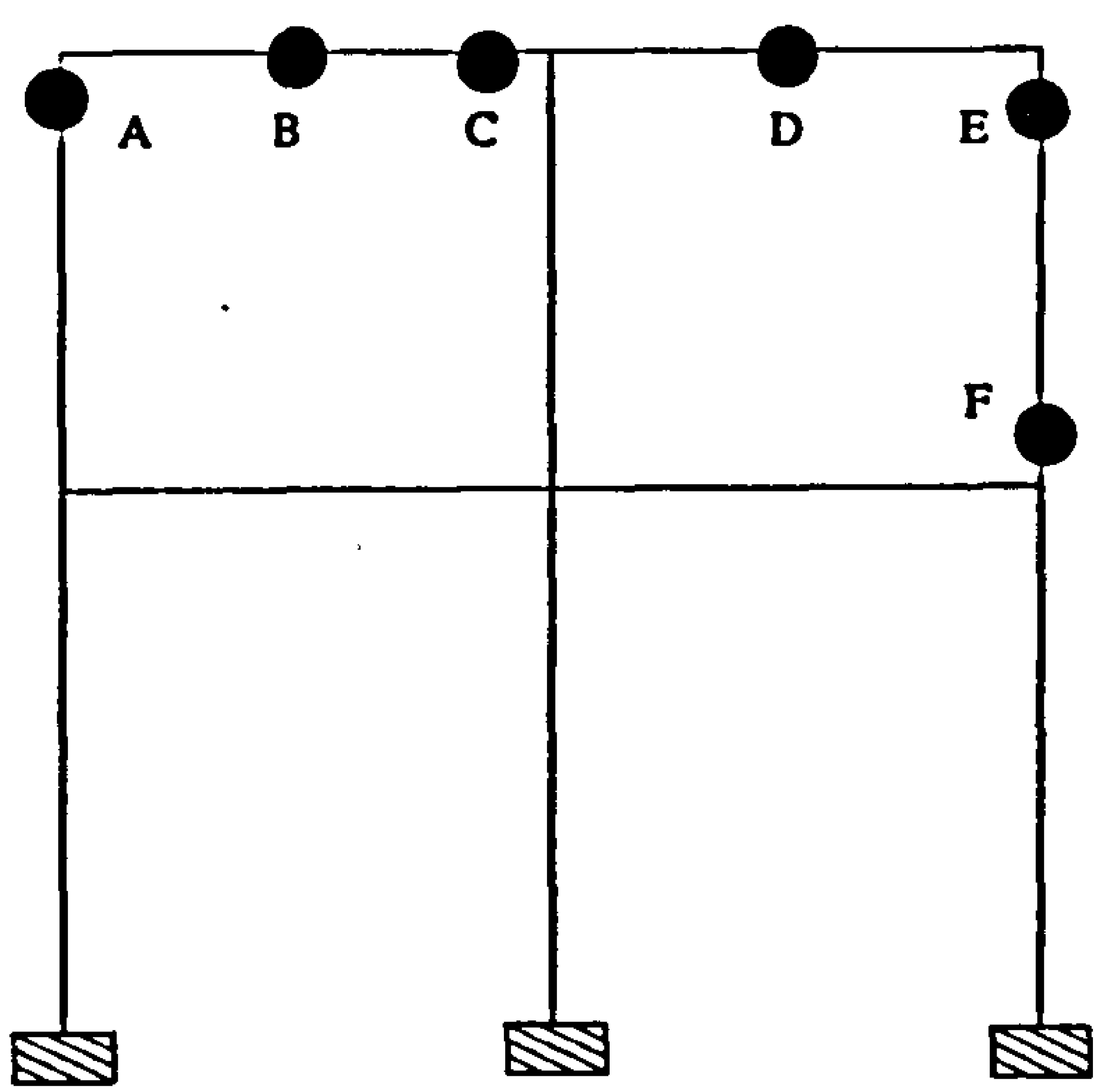
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 1
Load Case 2

Semi-Rigid Frame (Section Designation II)



Rigid Frame (Section Designation II)



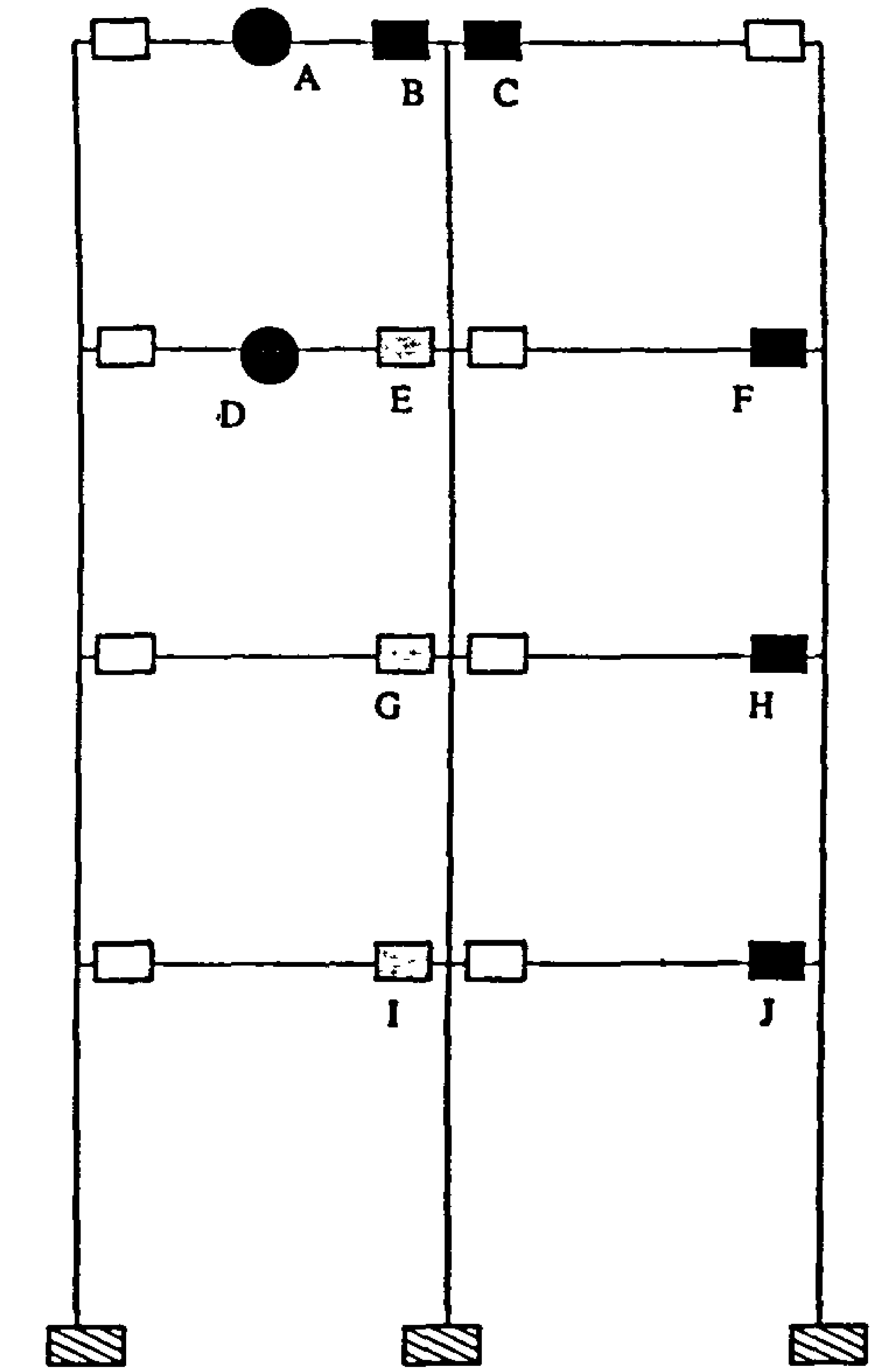
Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.87	2.64
B	1.78	2.37
C	1.78	2.11
D	1.83	2.43
E	1.83	2.58
F	1.78	2.64
G	1.83	N/A

Key:

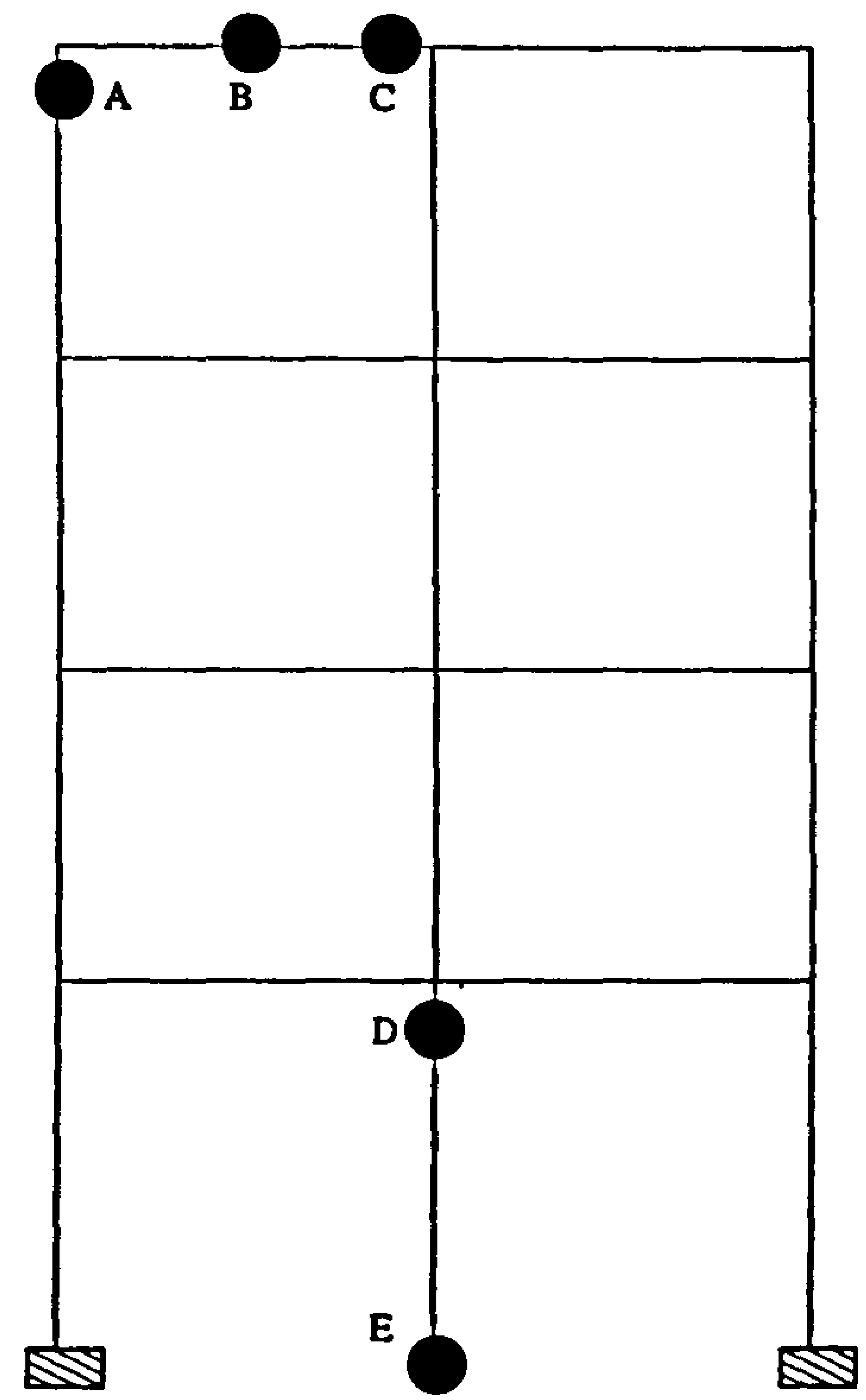
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 1
Load Case 3

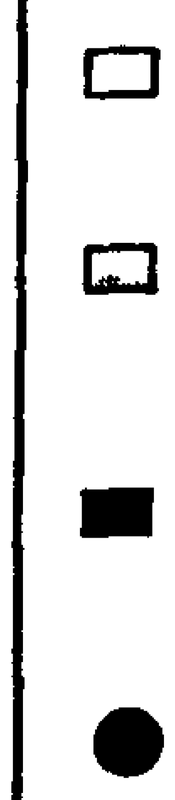
Semi-Rigid Frame (Section Designation II)



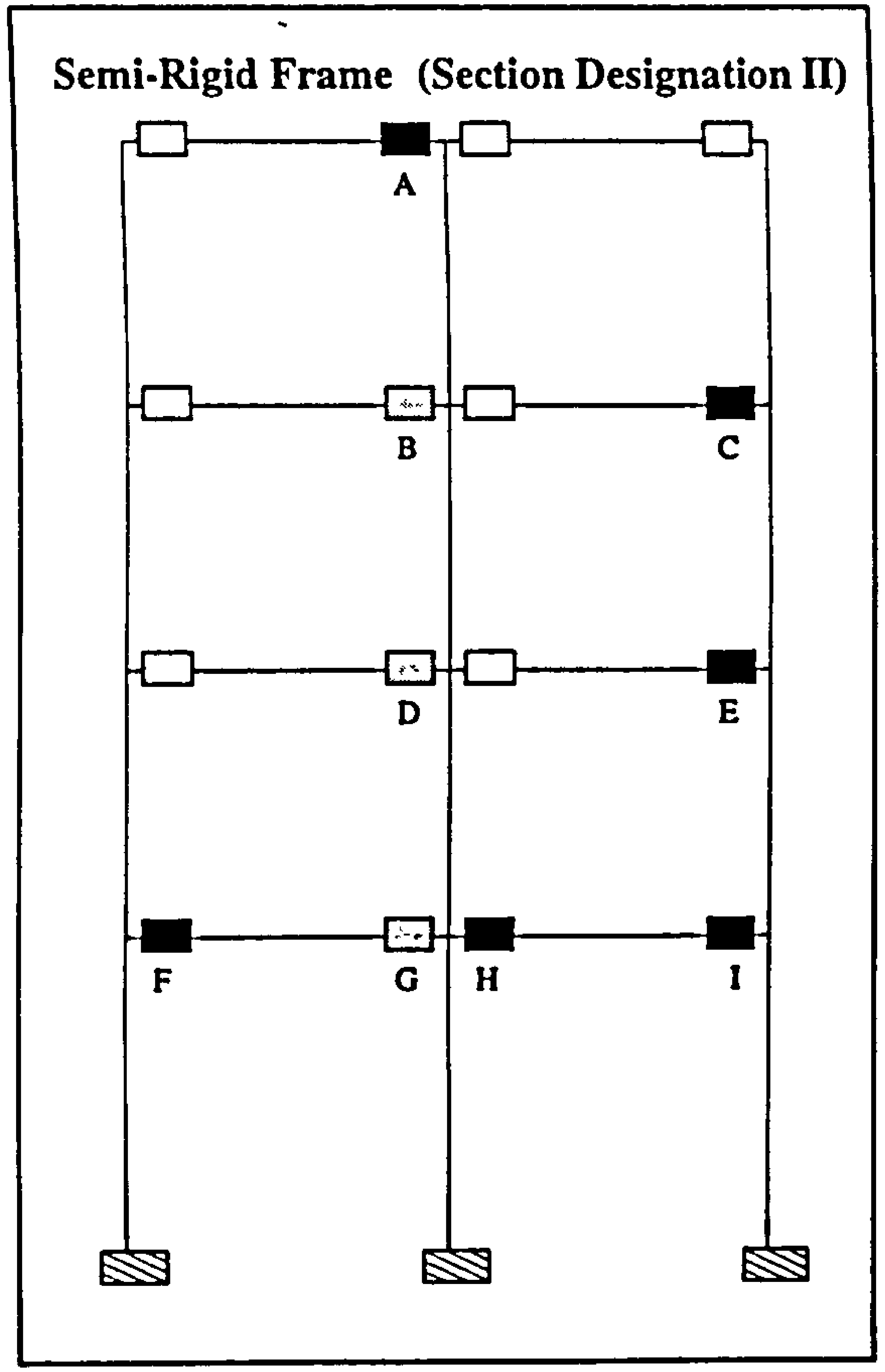
Rigid Frame (Section Designation II)

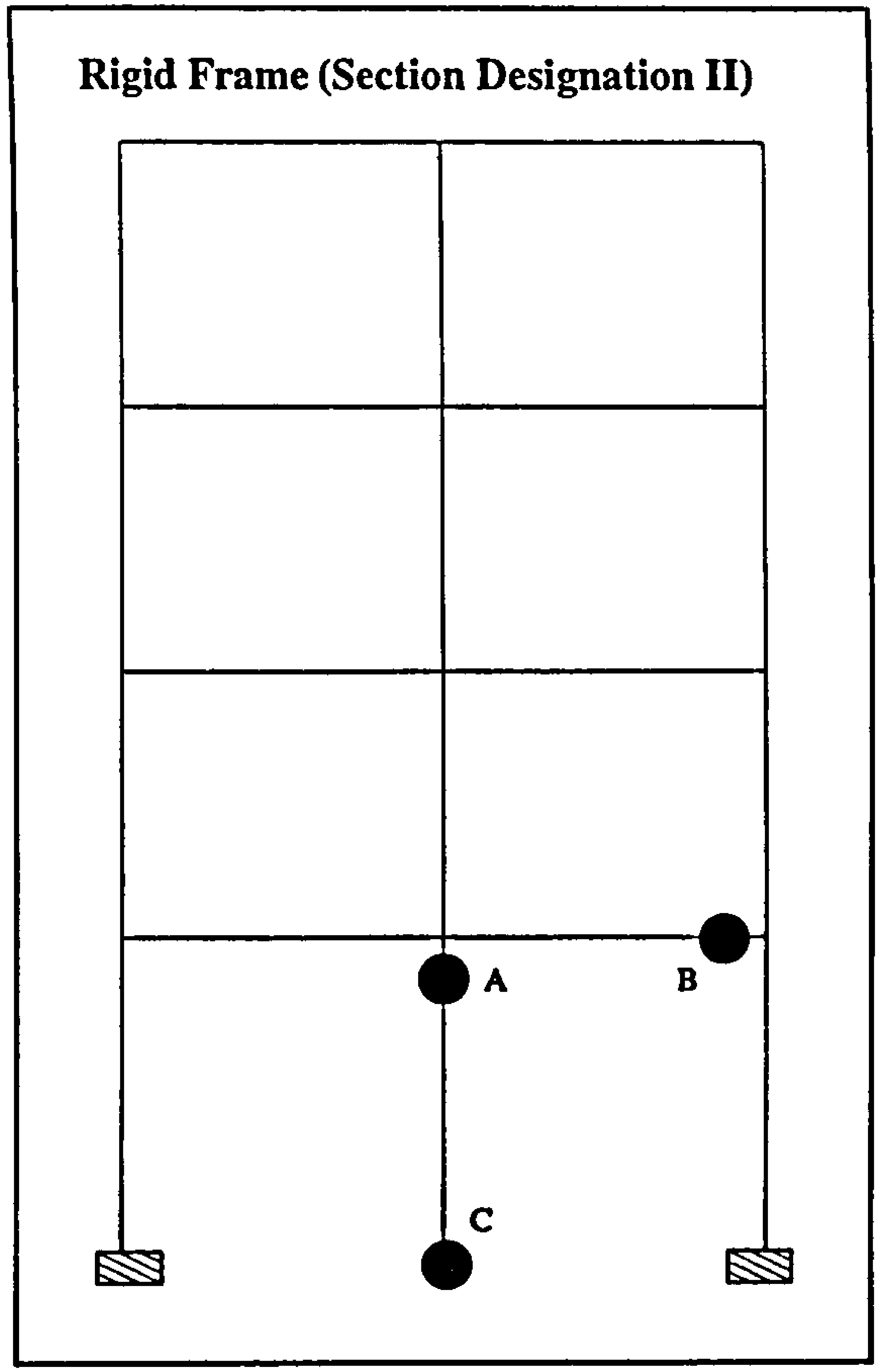


Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.35	1.83
B	1.35	1.70
C	1.35	1.66
D	1.40	1.78
E	< 1.00	1.77
F	1.40	N/A
G	< 1.00	N/A
H	1.35	N/A
I	< 1.00	N/A
J	1.35	N/A



Frame 2
Load Case 1





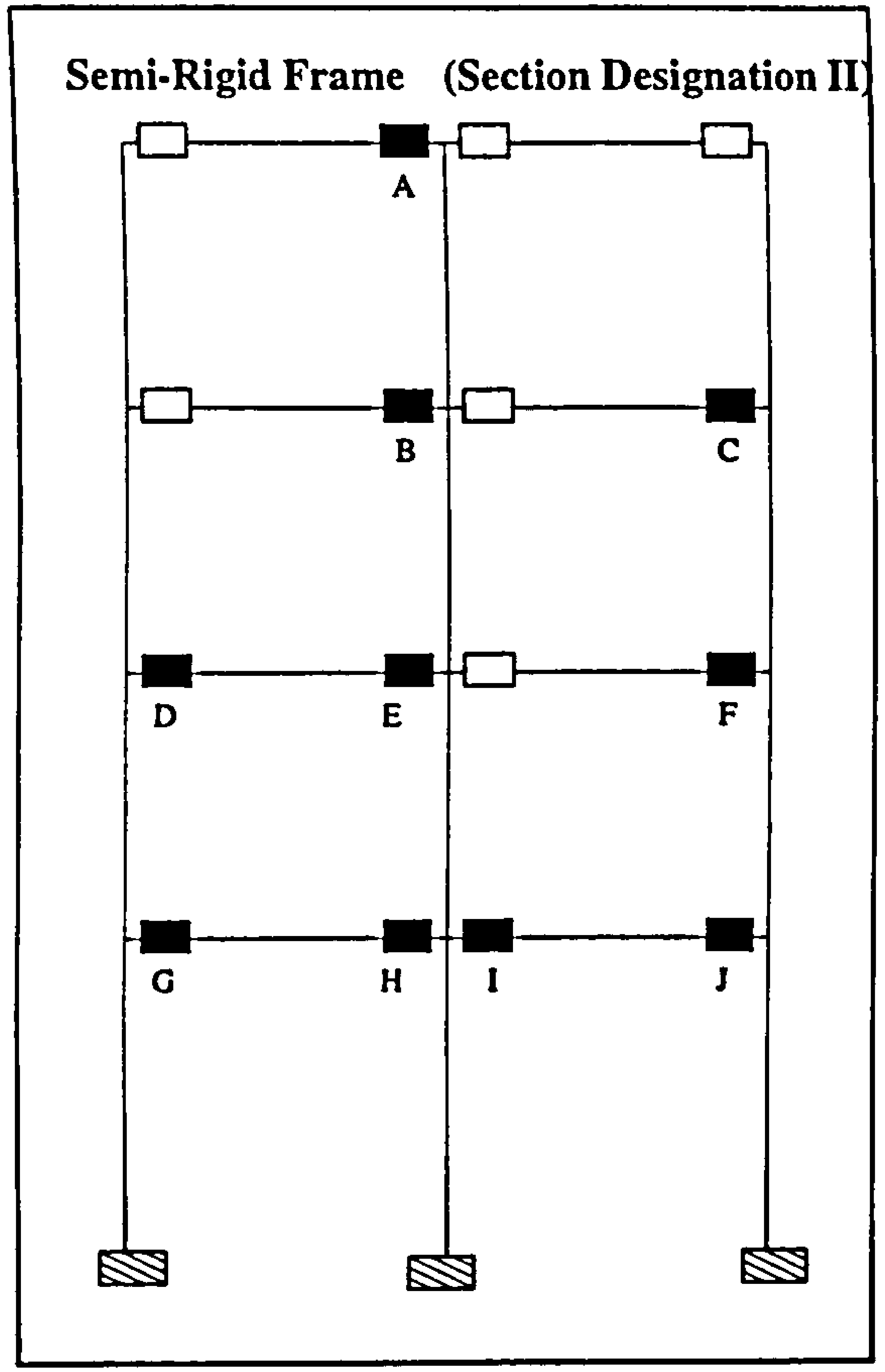
Frame 2
Load Case 2

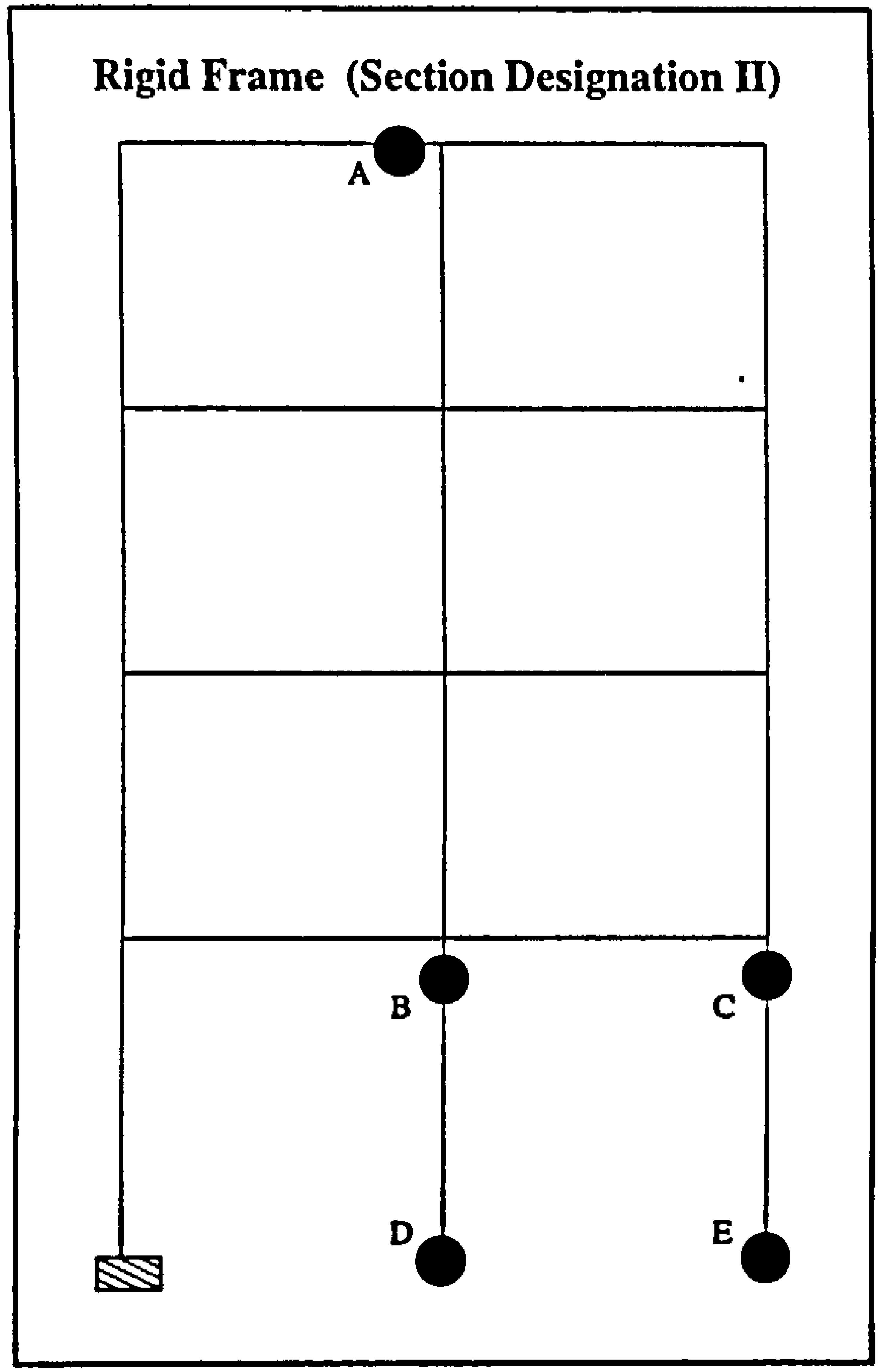
Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.46	1.75
B	< 1.00	1.83
C	1.46	1.72
D	< 1.00	N/A
E	1.46	N/A
F	< 1.00	N/A
G	1.46	N/A
H	1.46	N/A
I	1.46	N/A

Key:

- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Plastic-hinges in minor -axis “wind-moment” frames.

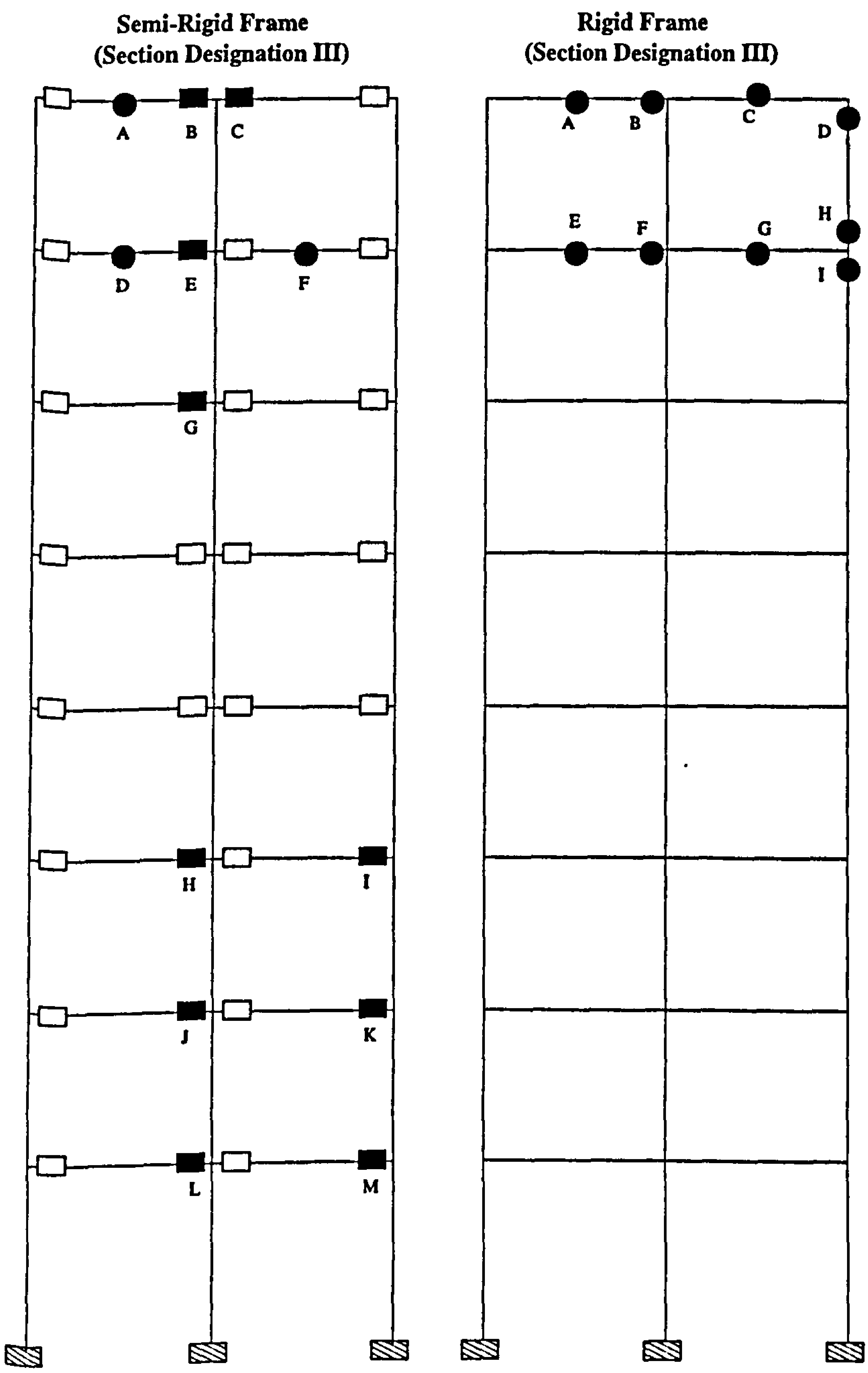




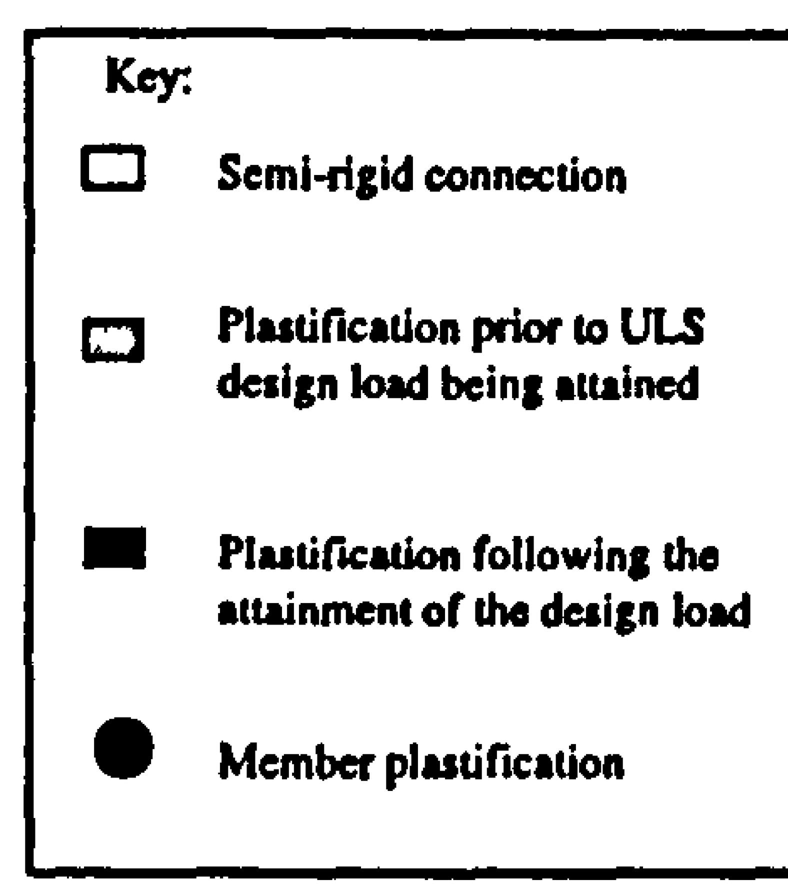
Frame 2
Load Case 3

Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.68	2.12
B	1.68	2.19
C	1.68	2.28
D	1.68	2.28
E	1.68	2.28
F	1.68	N/A
G	1.68	N/A
H	1.68	N/A
I	1.68	N/A
J	1.68	N/A

Key:
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

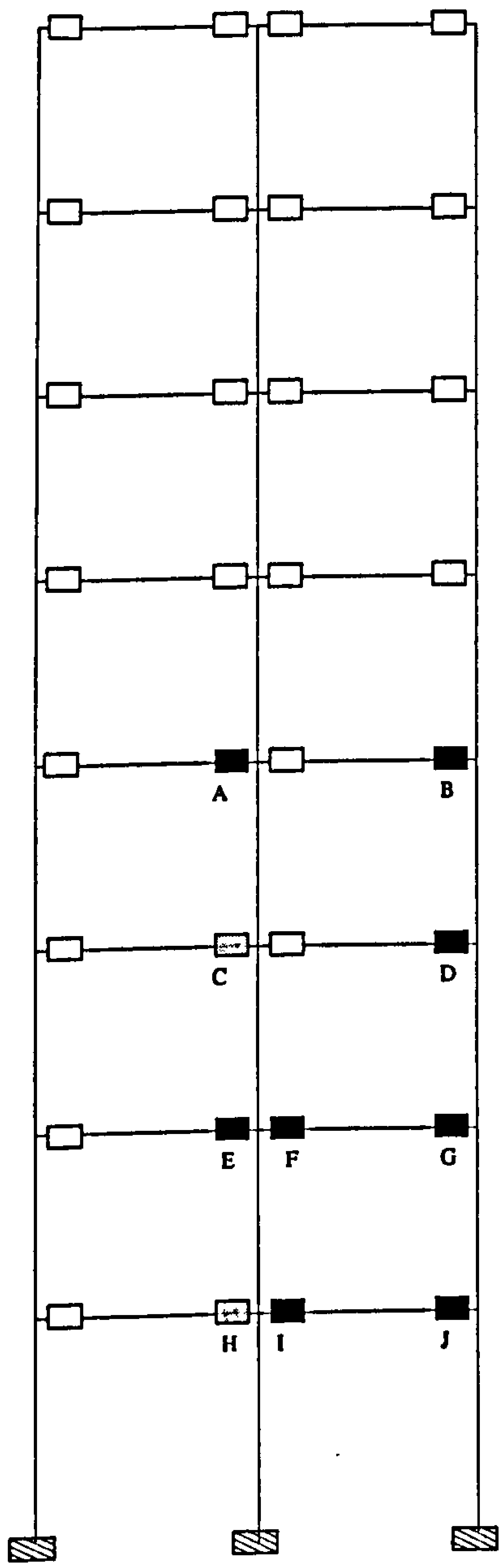


Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.56	1.96
B	1.54	1.83
C	1.54	1.97
D	1.54	2.06
E	1.54	1.92
F	1.56	1.83
G	1.56	1.92
H	1.54	2.05
I	1.56	2.07
J	1.54	N/A
K	1.62	N/A
L	1.54	N/A
M	1.62	N/A

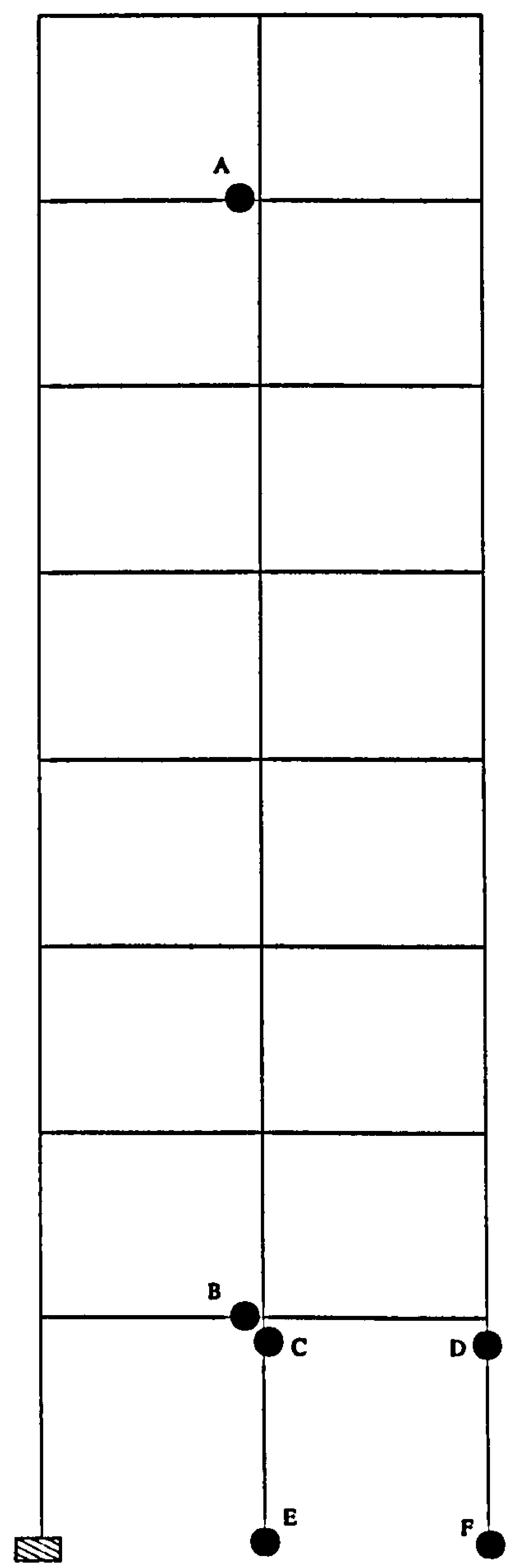


Frame 3
Load Case 1

Semi-Rigid Frame
(Section Designation III)



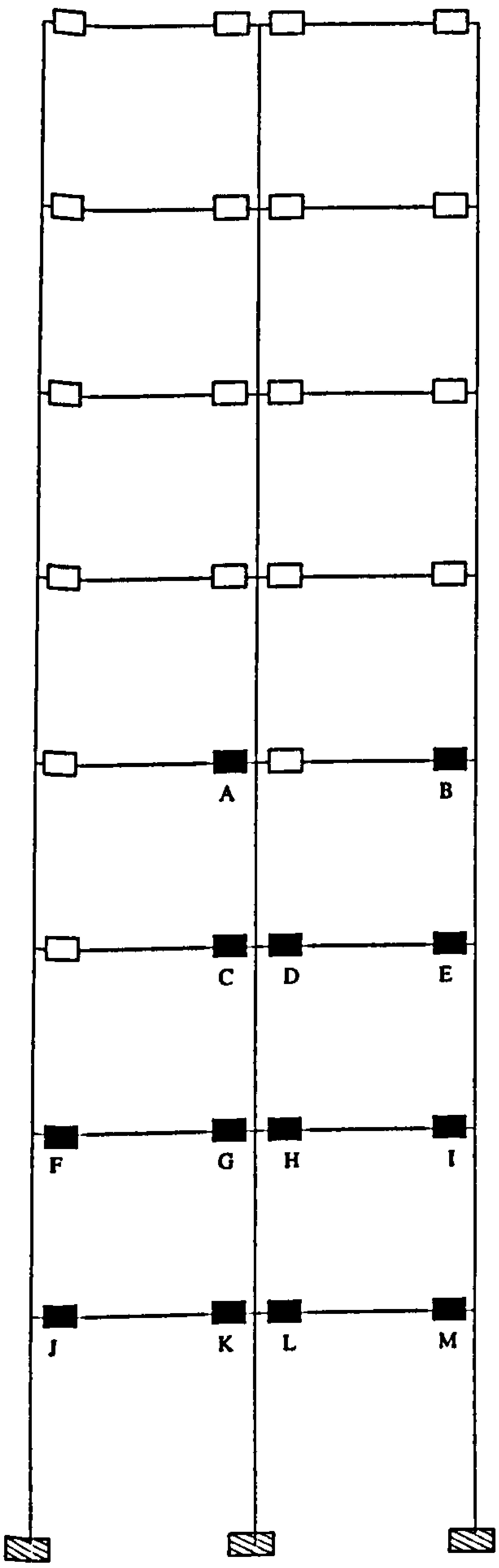
Rigid Frame
(Section Designation III)

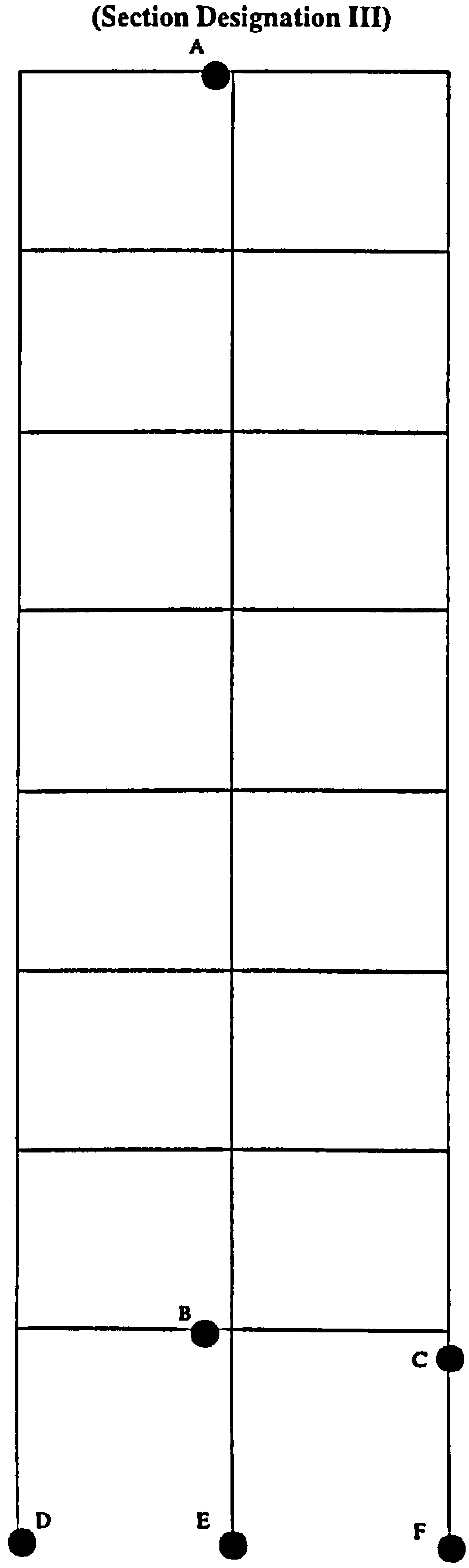


Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.23	1.93
B	1.23	1.79
C	< 1.00	1.97
D	1.23	1.95
E	1.23	1.90
F	1.23	1.94
G	1.23	N/A
H	< 1.00	N/A
I	1.23	N/A
J	1.23	N/A

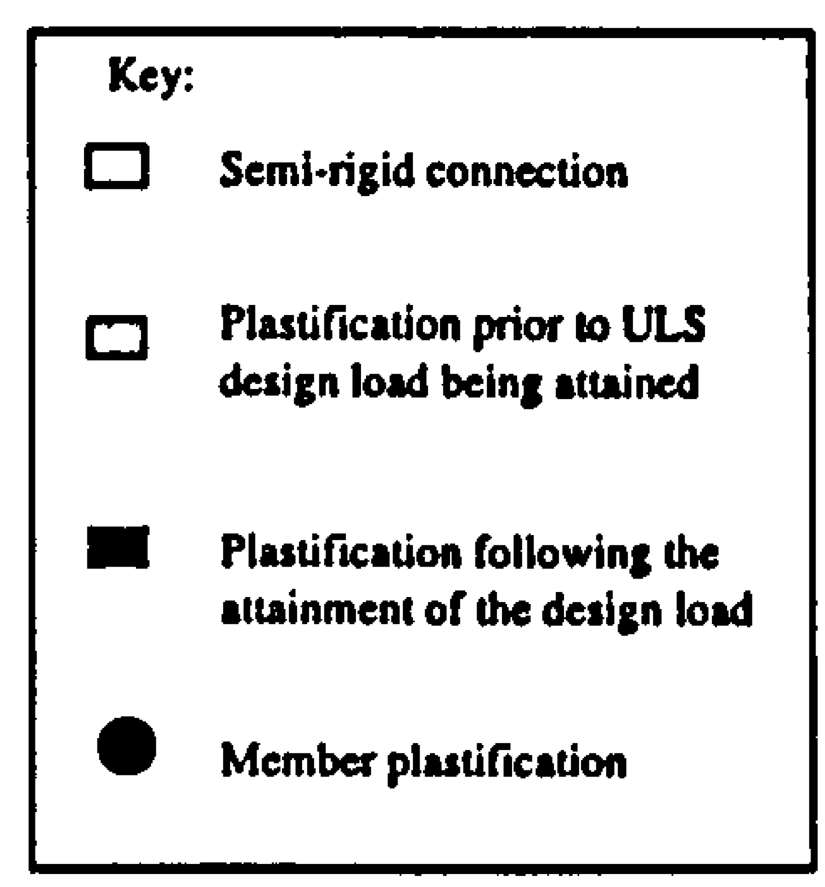
Key:
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 3
Load Case 2



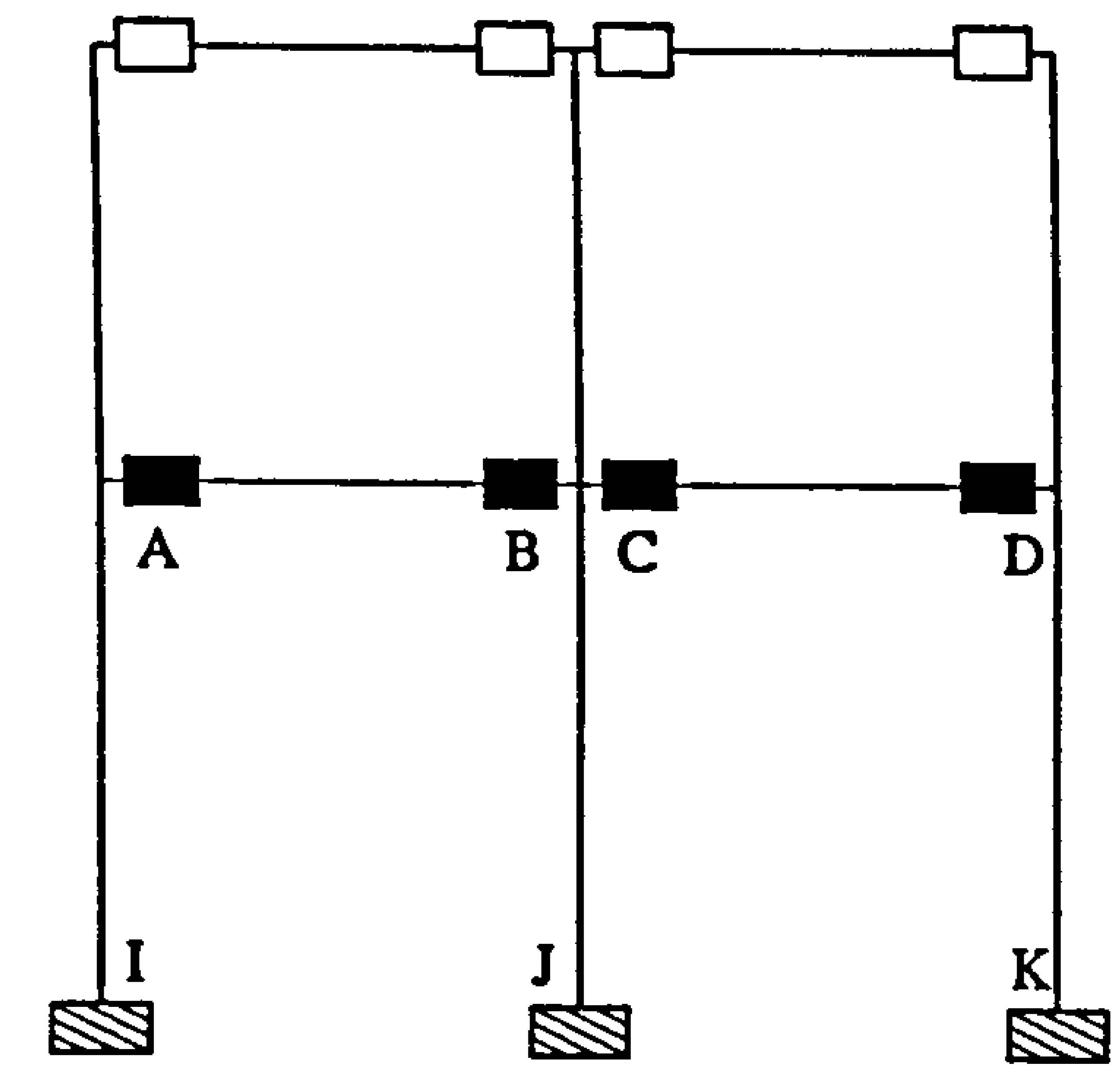


Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.23	2.23
B	1.23	2.04
C	1.23	2.25
D	1.23	2.27
E	1.23	2.16
F	1.23	2.19
G	1.23	N/A
H	1.23	N/A
I	1.23	N/A
J	1.23	N/A
K	1.23	N/A
L	1.23	N/A
M	1.23	N/A



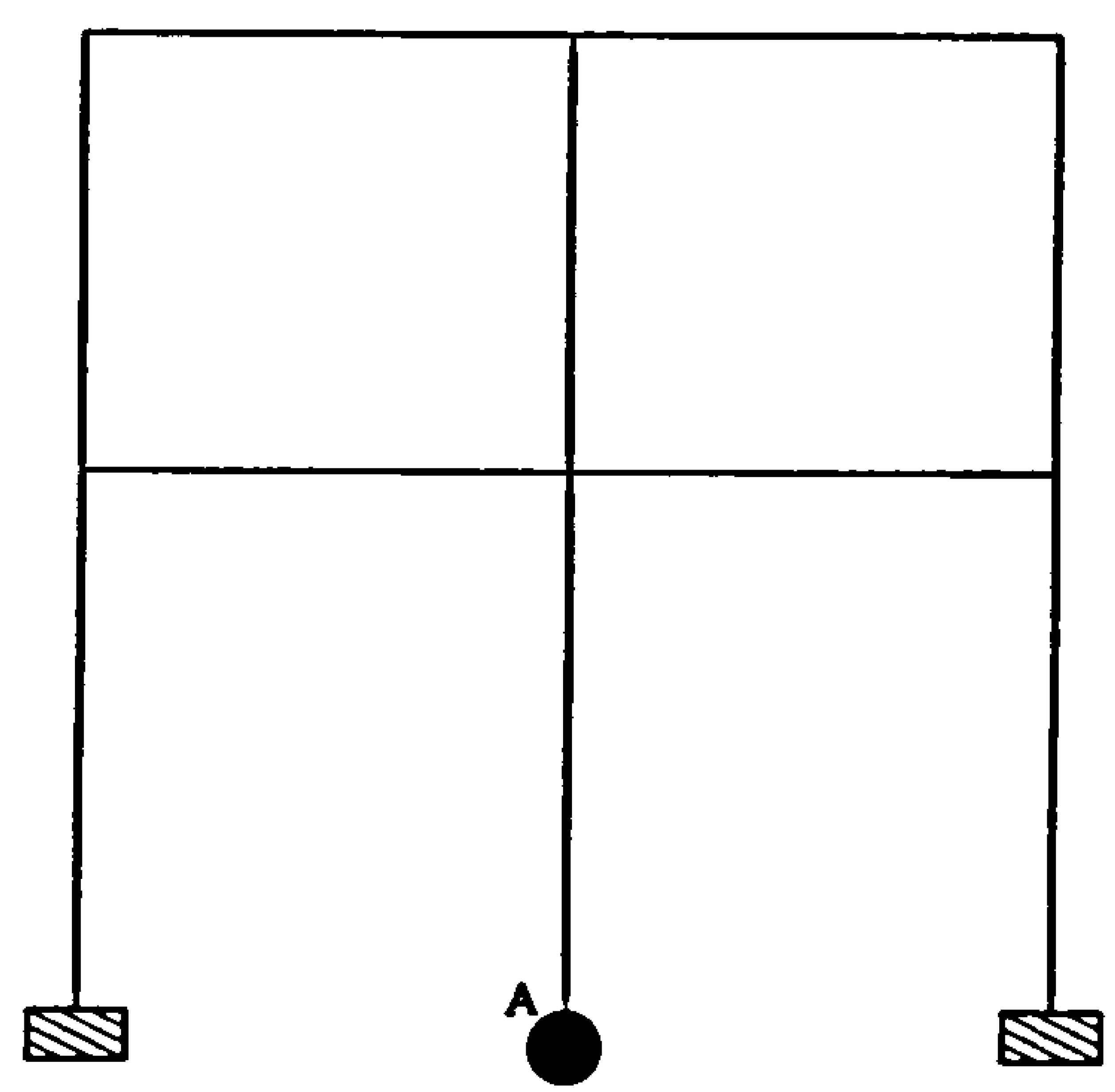
Frame 3
Load Case 3

Semi-Rigid Frame (Section Designation II)



Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.23	2.07
B	1.23	N/A
C	1.23	N/A
D	1.23	N/A

Rigid Frame (Section Designation II)



Key:

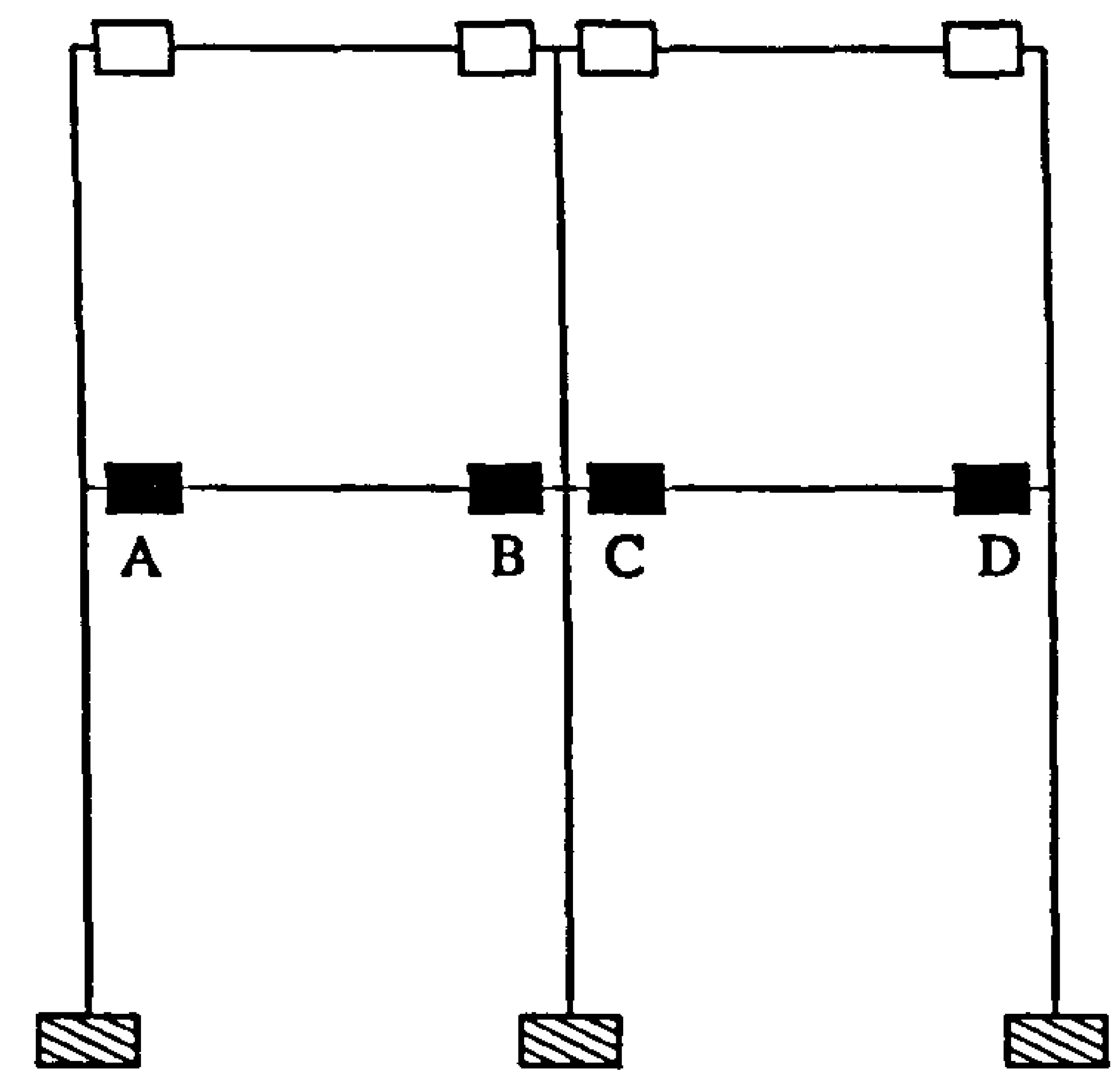
Semi-rigid connection

Plastification prior to ULS design load being attained

Plastification following the attainment of the design load

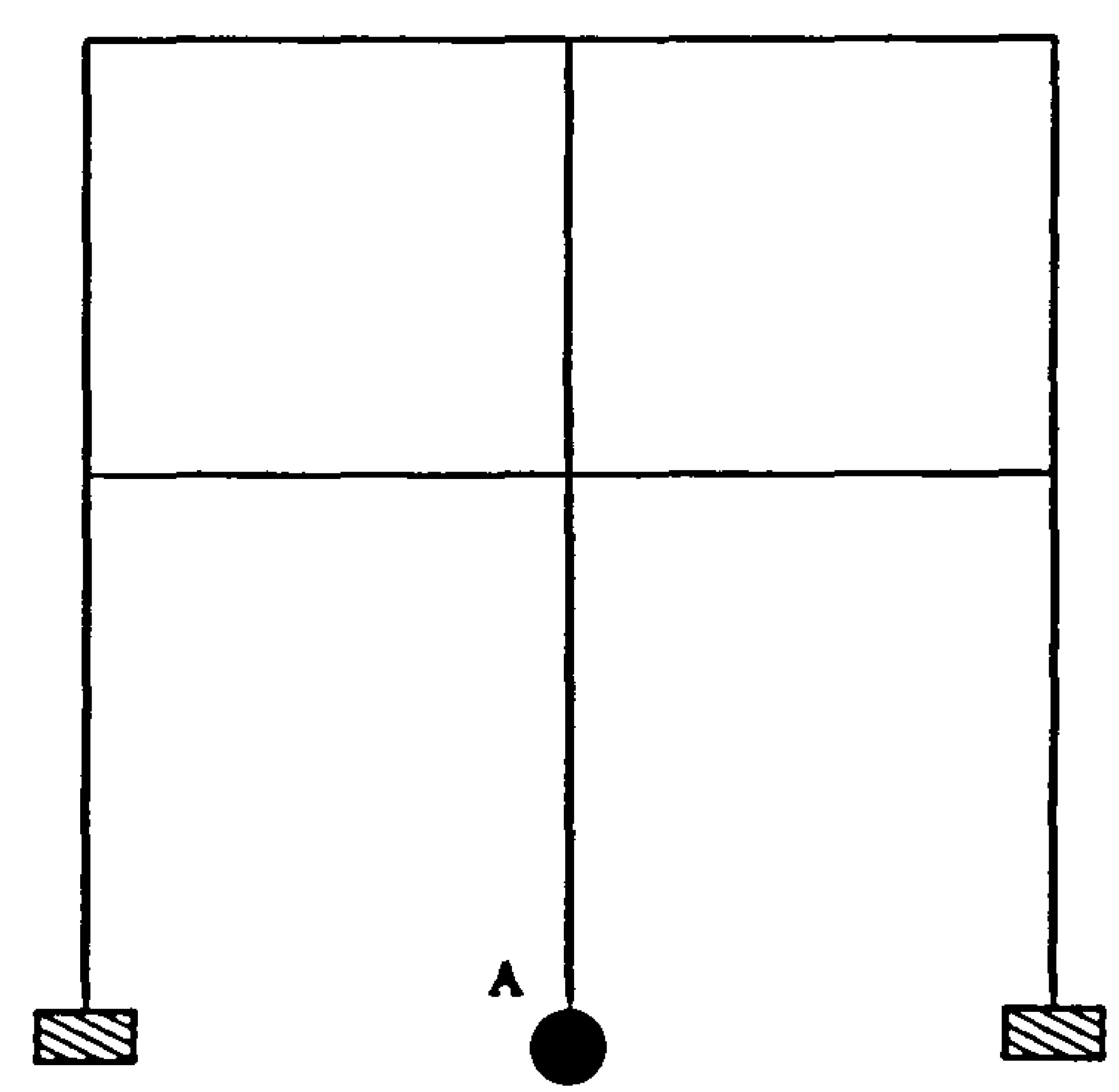
Member plastification

Frame 4
Load Case 1



Semi-Rigid Frame (Section Designation II)

Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.16	1.96
B	1.16	N/A
C	1.16	N/A
D	1.16	N/A



Rigid Frame (Section Designation II)

Key:

Semi-rigid connection

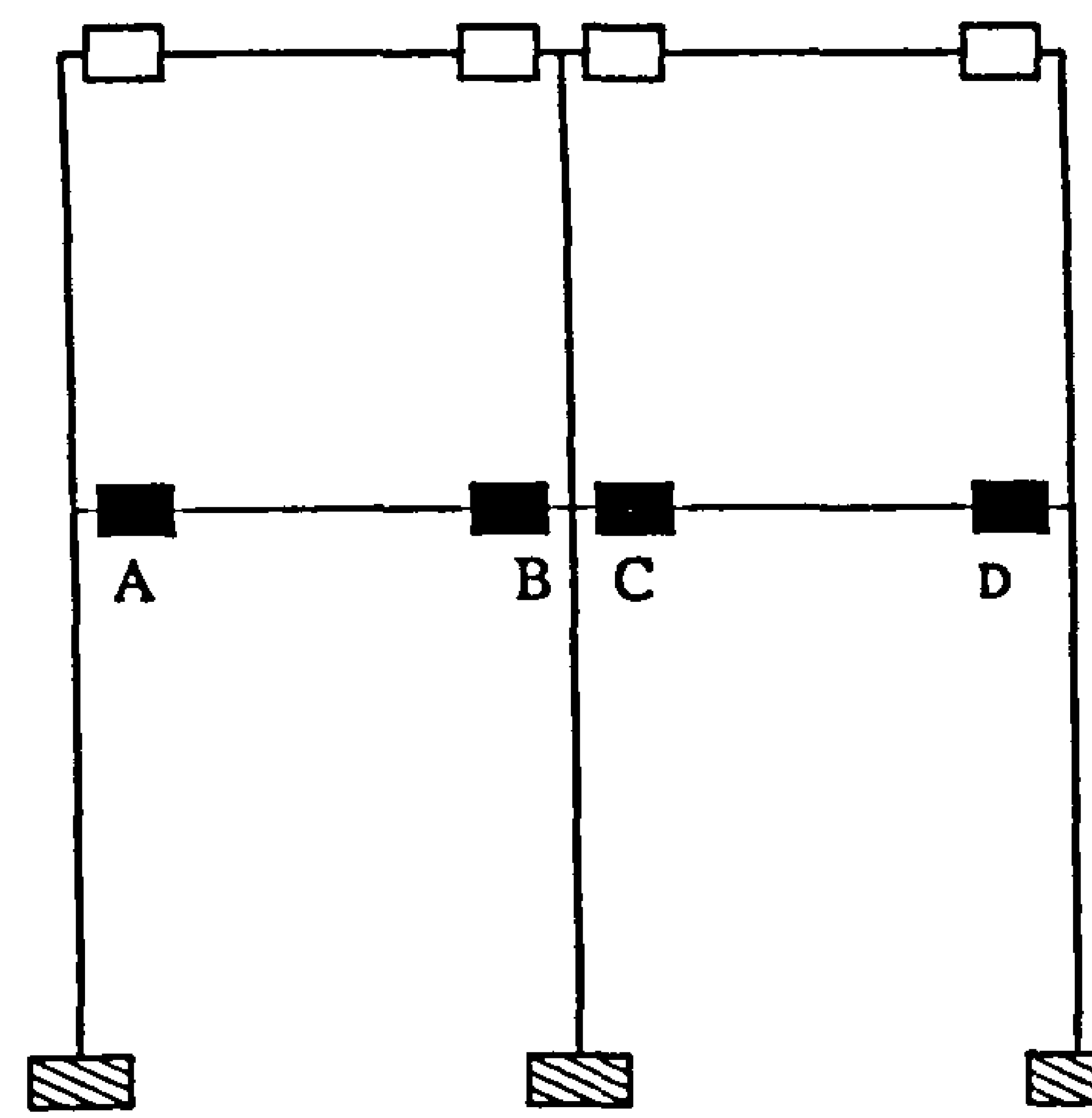
Plastification prior to ULS design load being attained

Plastification following the attainment of the design load

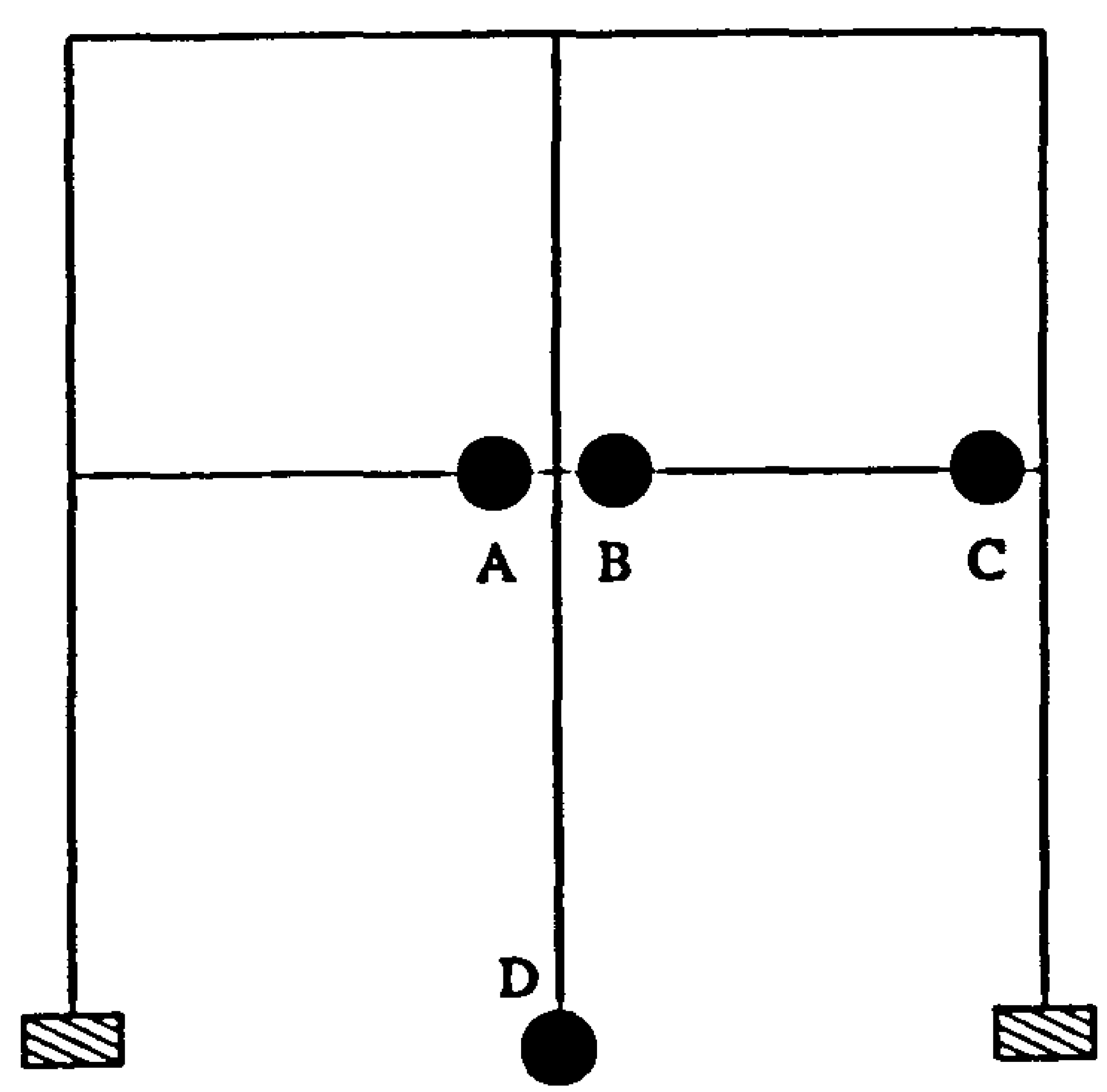
Member plastification

Frame 4
Load Case 2

Semi-Rigid Frame (Section Designation II)



Rigid Frame (Section Designation II)



Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.37	2.31
B	1.37	2.30
C	1.37	2.34
D	1.37	2.35

Key:

Semi-rigid connection

Plastification prior to ULS design load being attained

Plastification following the attainment of the design load

Member plastification

Frame 4
Load Case 3

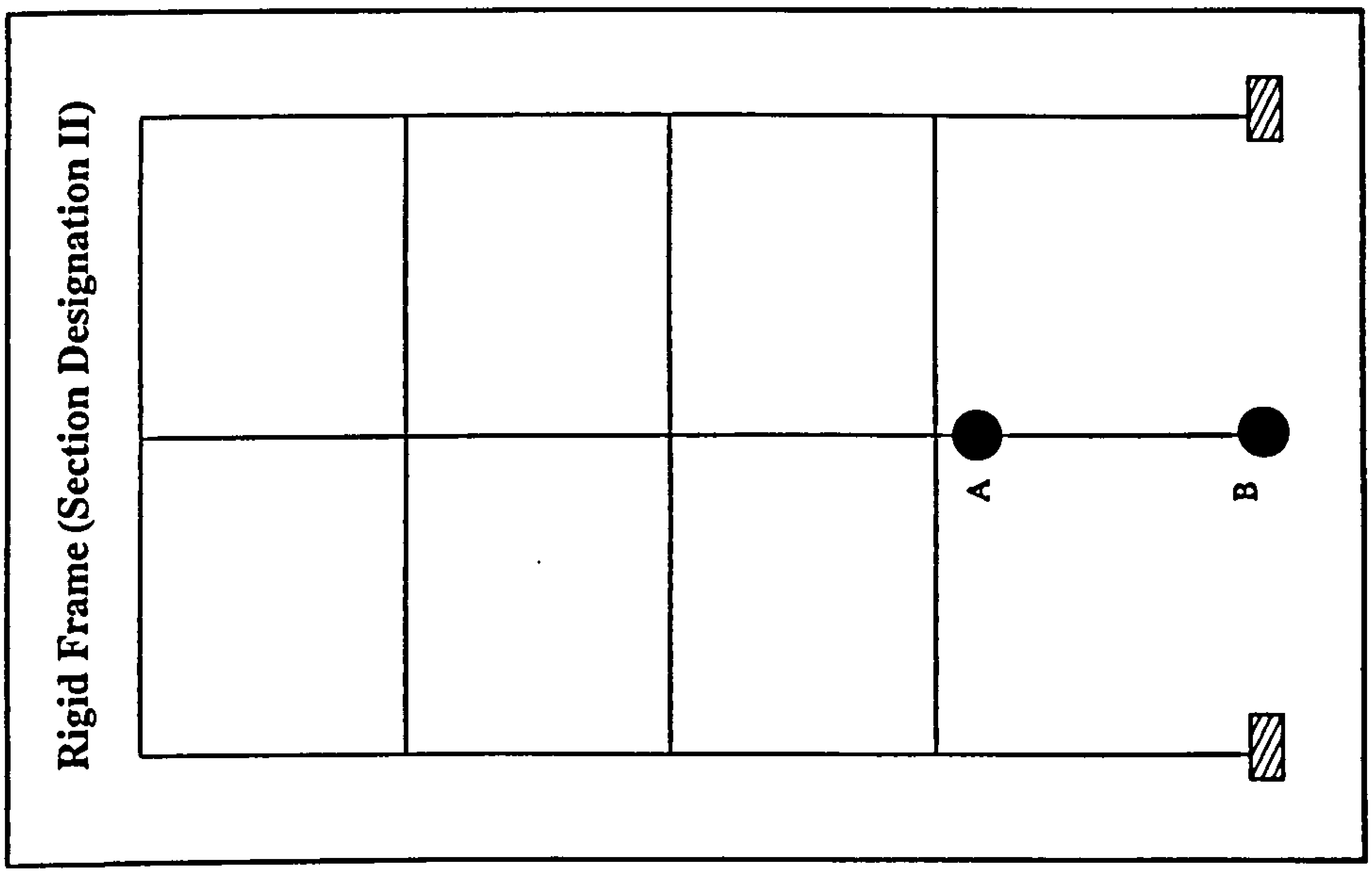
Plastic-hinges in minor -axis "wind-moment" frames.

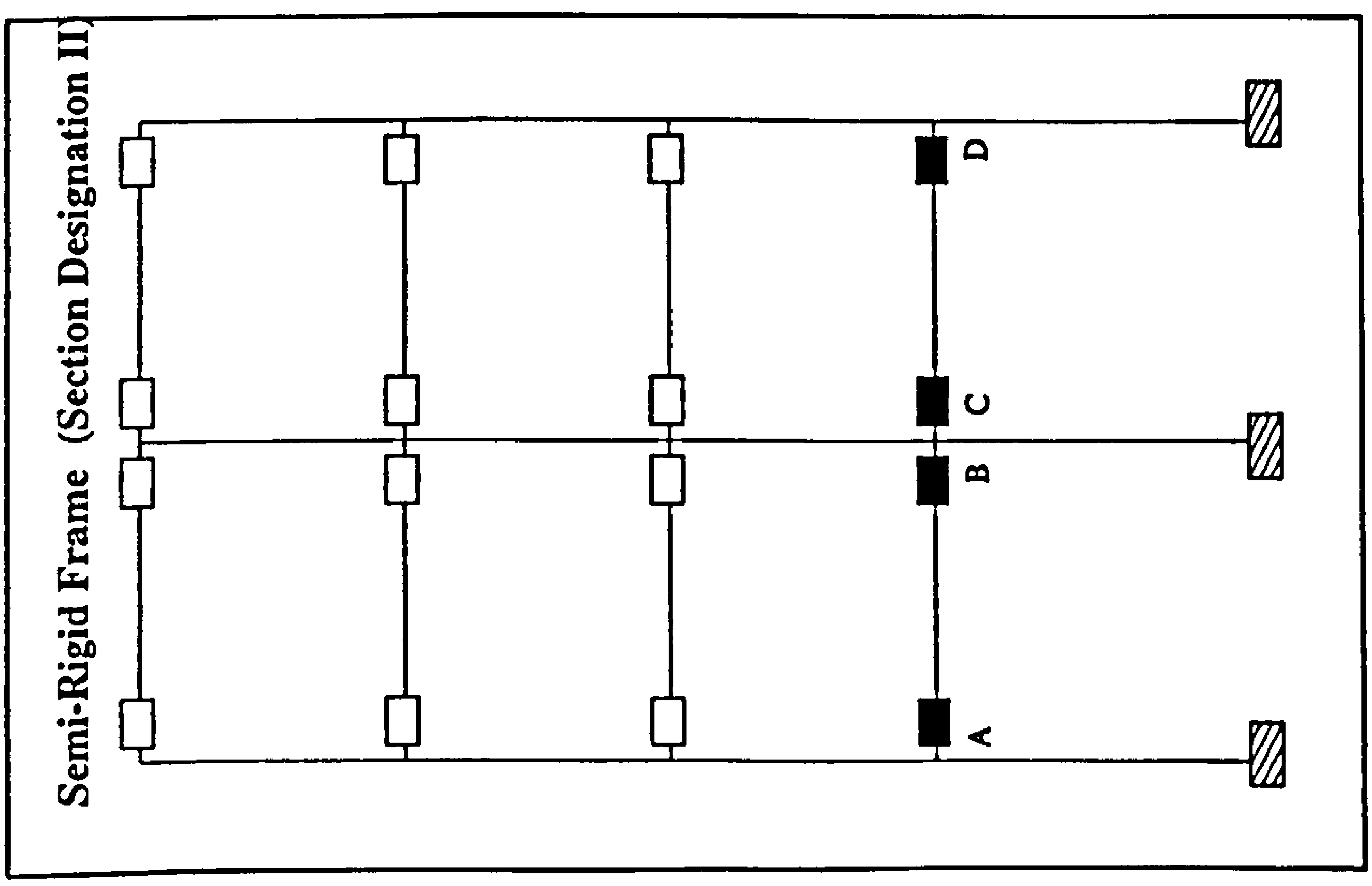
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.37	1.85
B	1.37	1.82
C	1.37	N/A
D	1.37	N/A

Key:

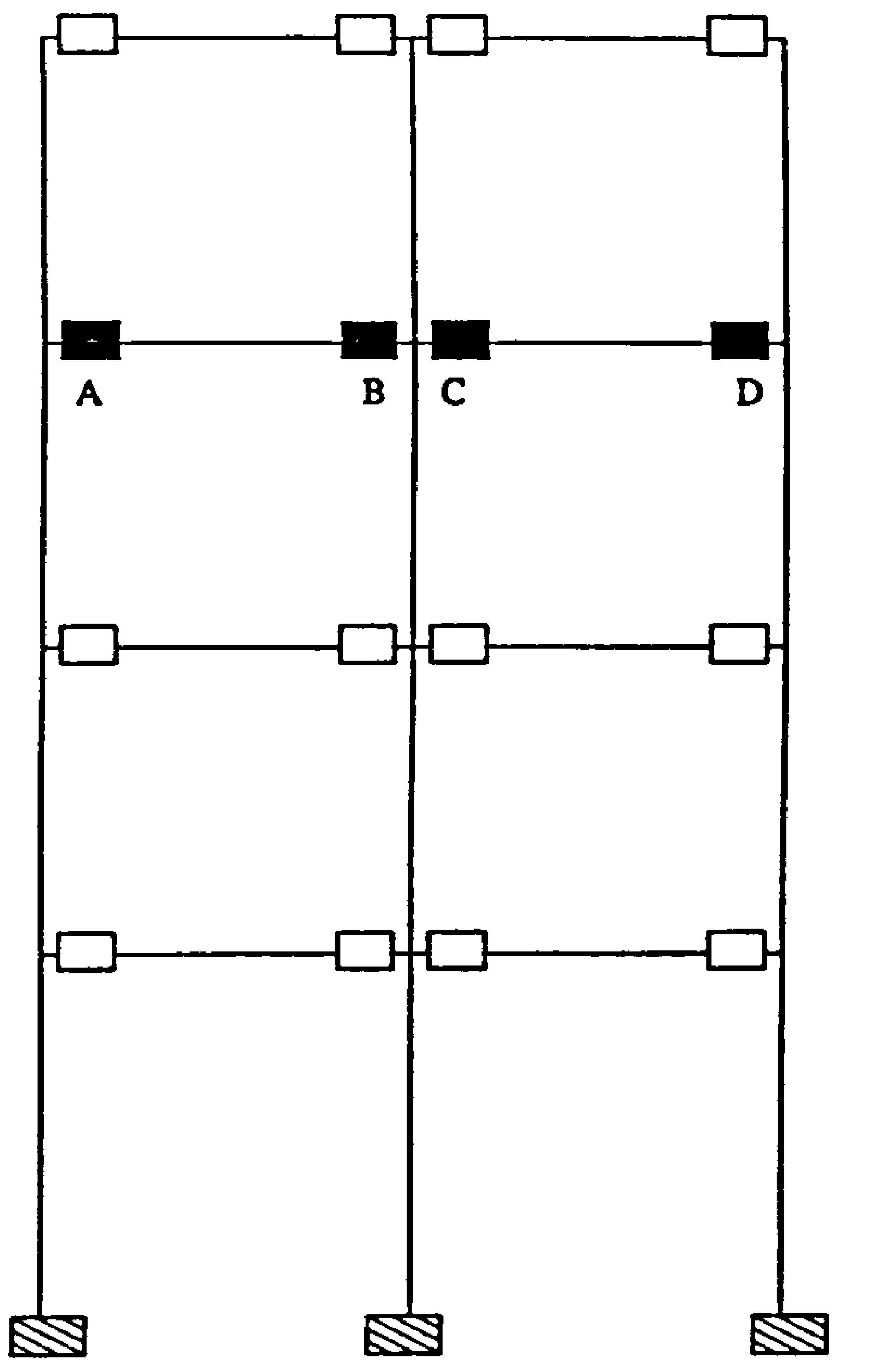
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 5
Load Case 1

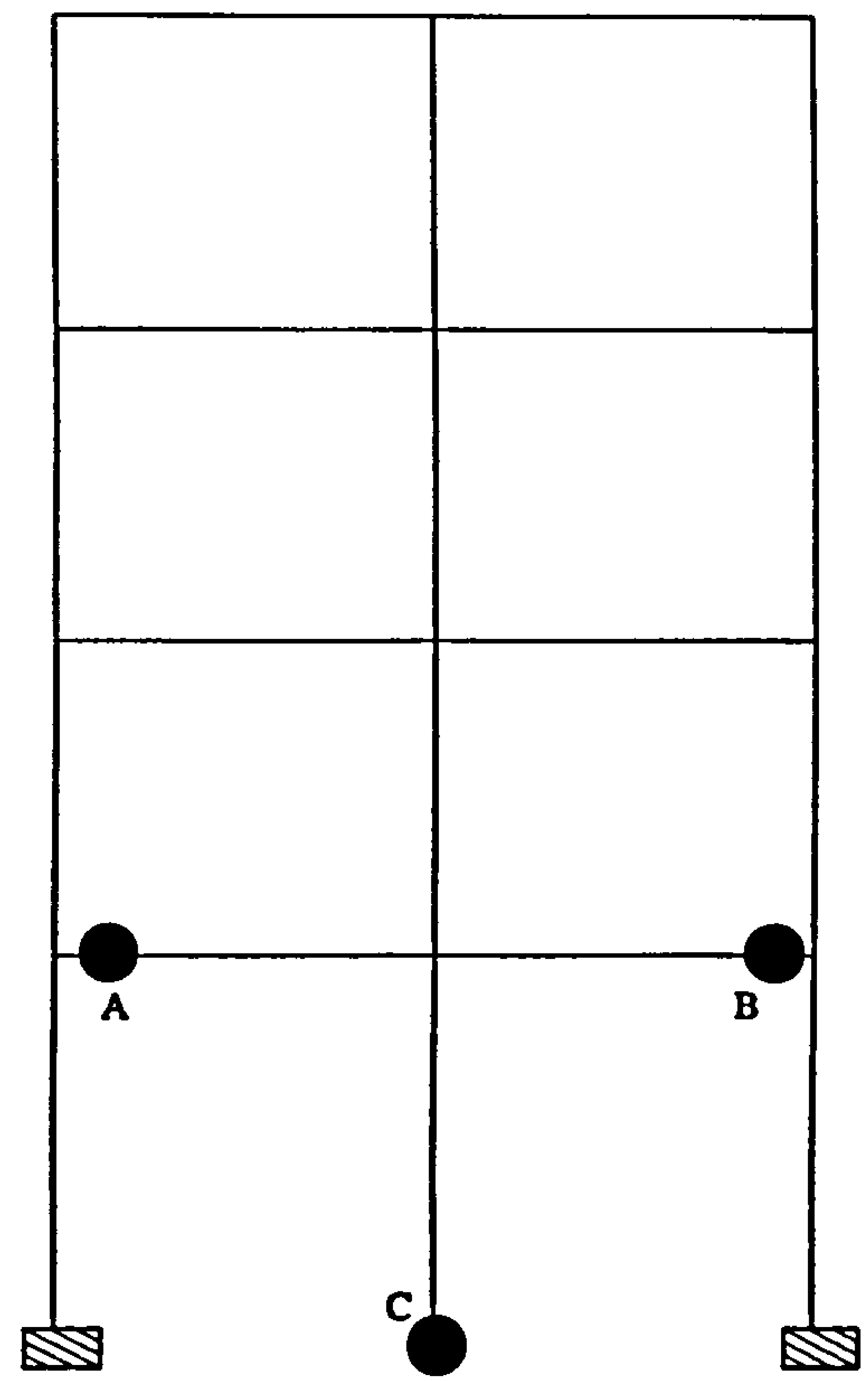




Semi-Rigid Frame (Section Designation II)



Rigid Frame (Section Designation II)



Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.09	1.77
B	1.09	1.76
C	1.09	1.67
D	1.09	N/A

Key:

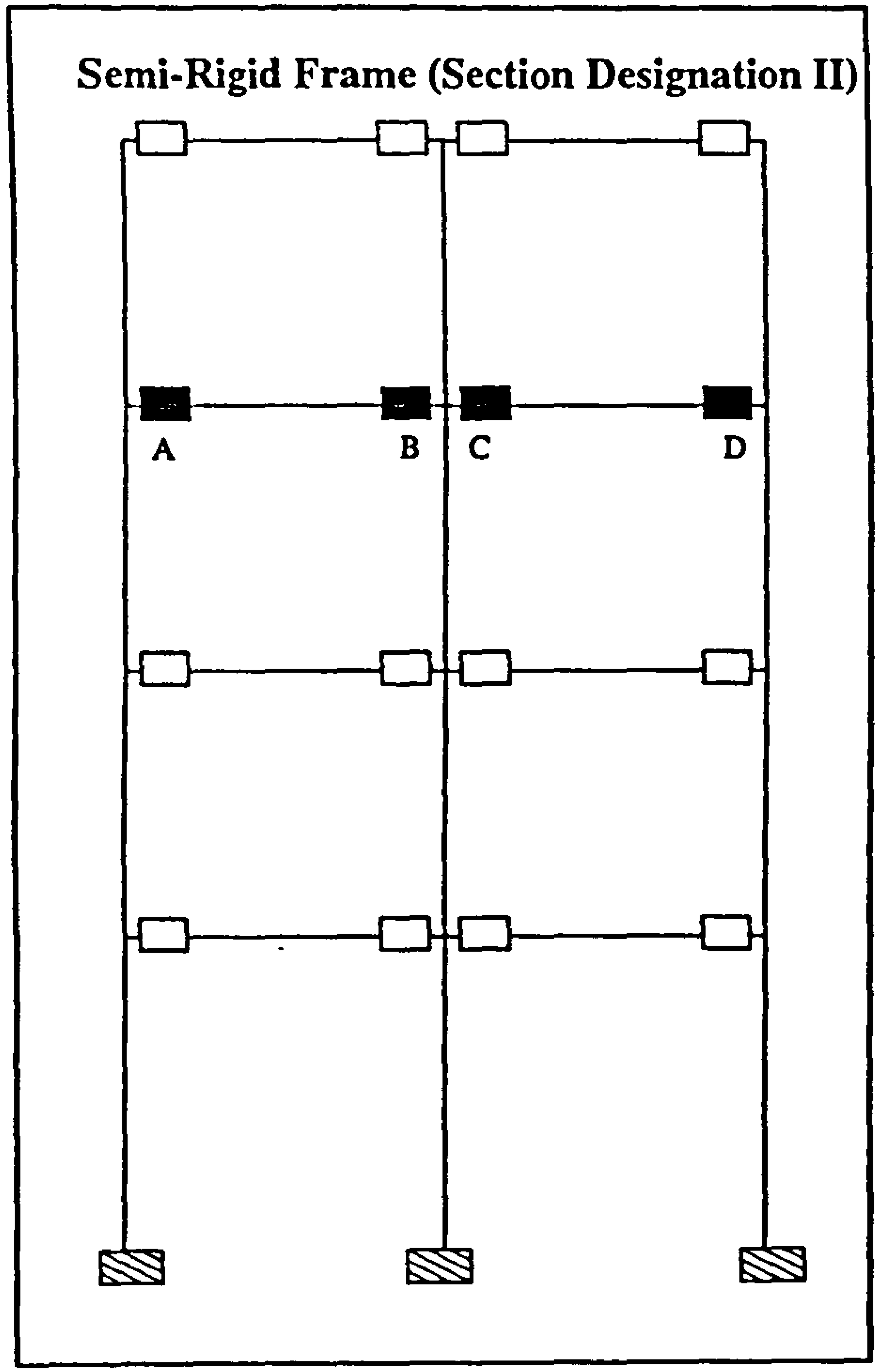
Semi-rigid connection

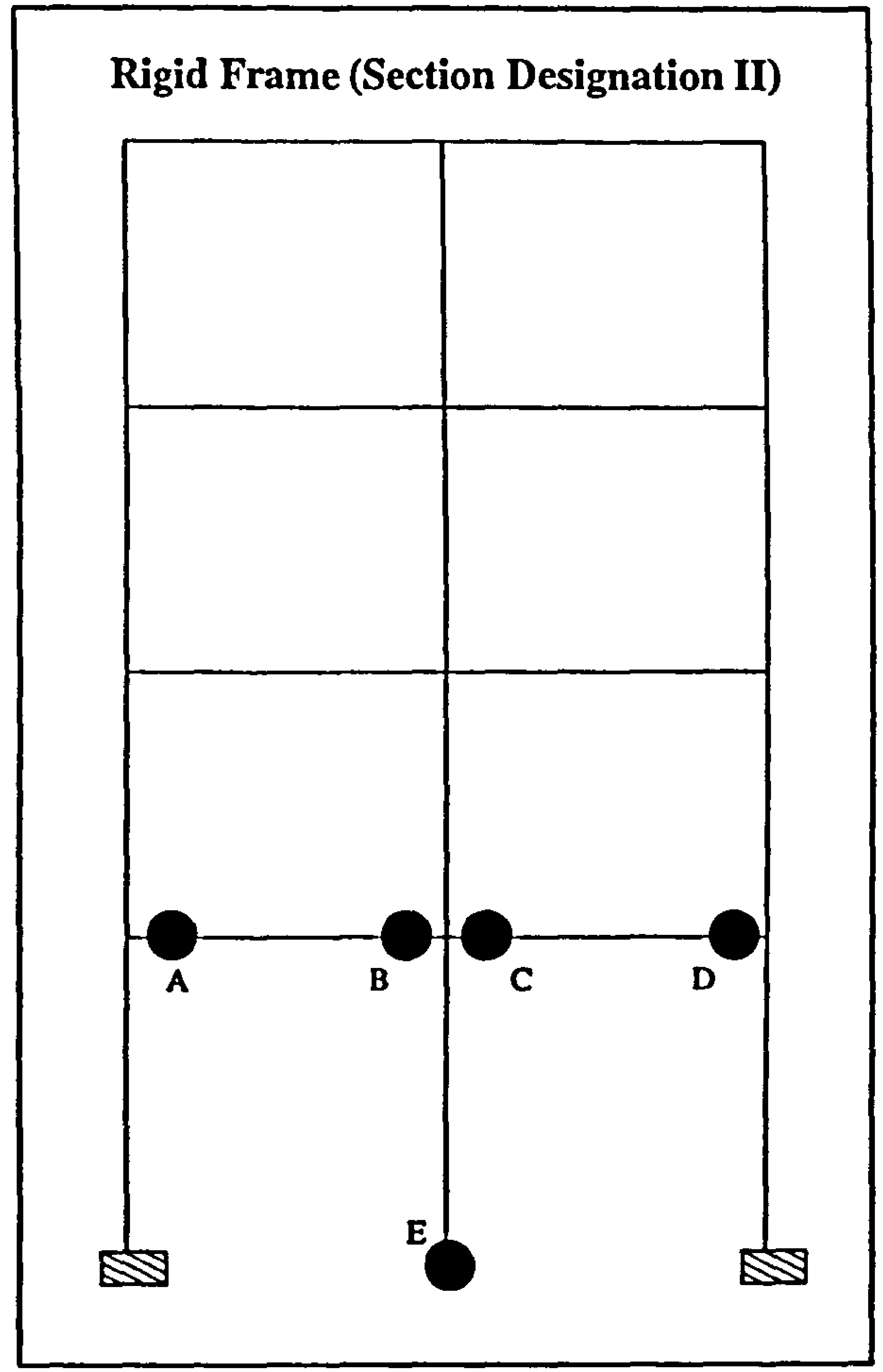
Plastification prior to ULS design load being attained

Plastification following the attainment of the design load

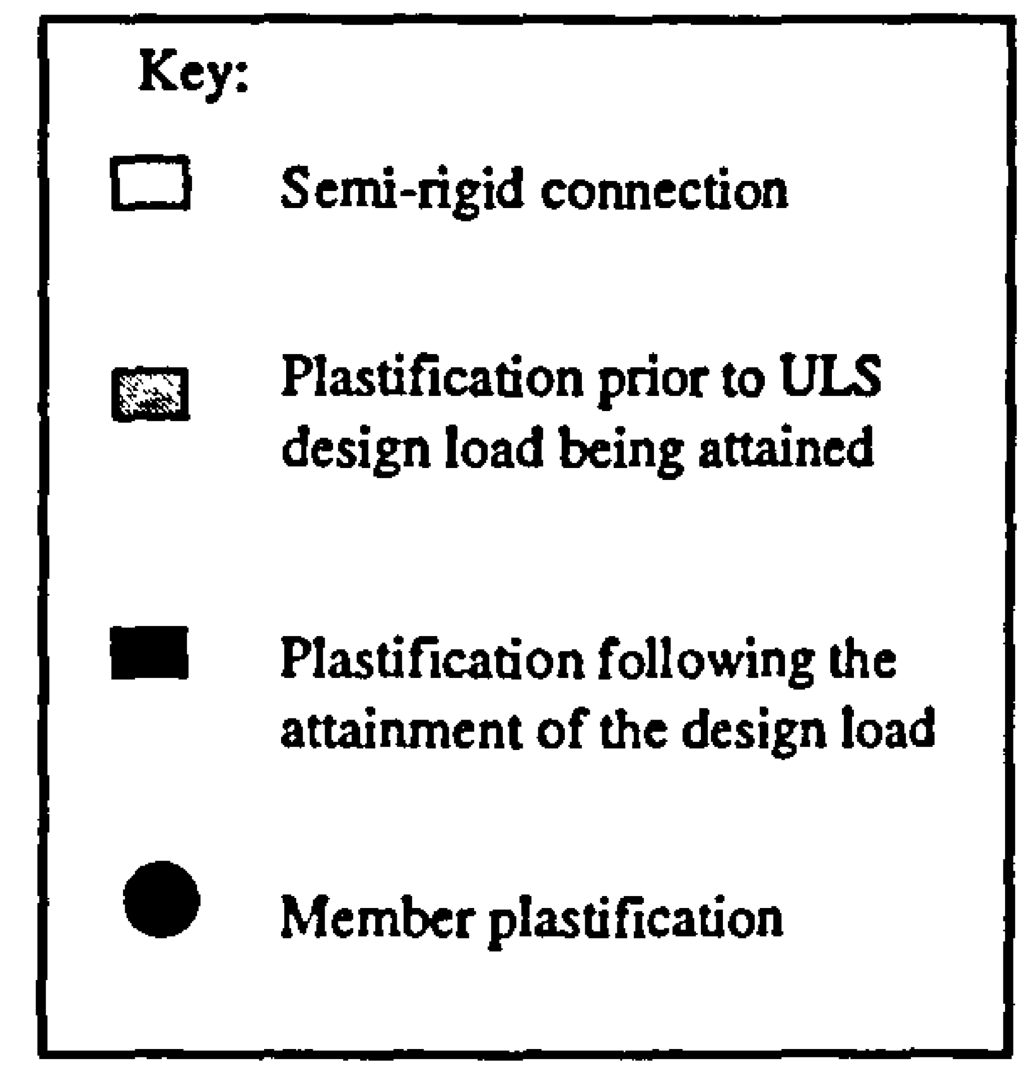
Member plastification

Frame 5
Load Case 2



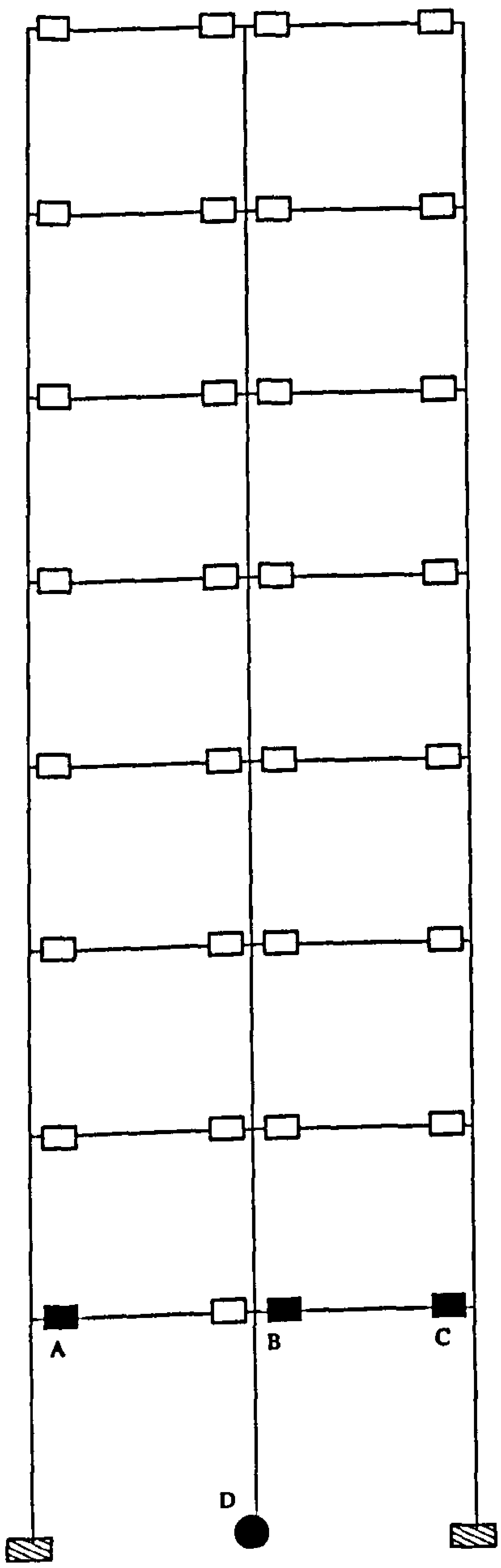


Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.18	2.02
B	1.18	2.02
C	1.18	2.00
D	1.18	2.00
E	N/A	1.98

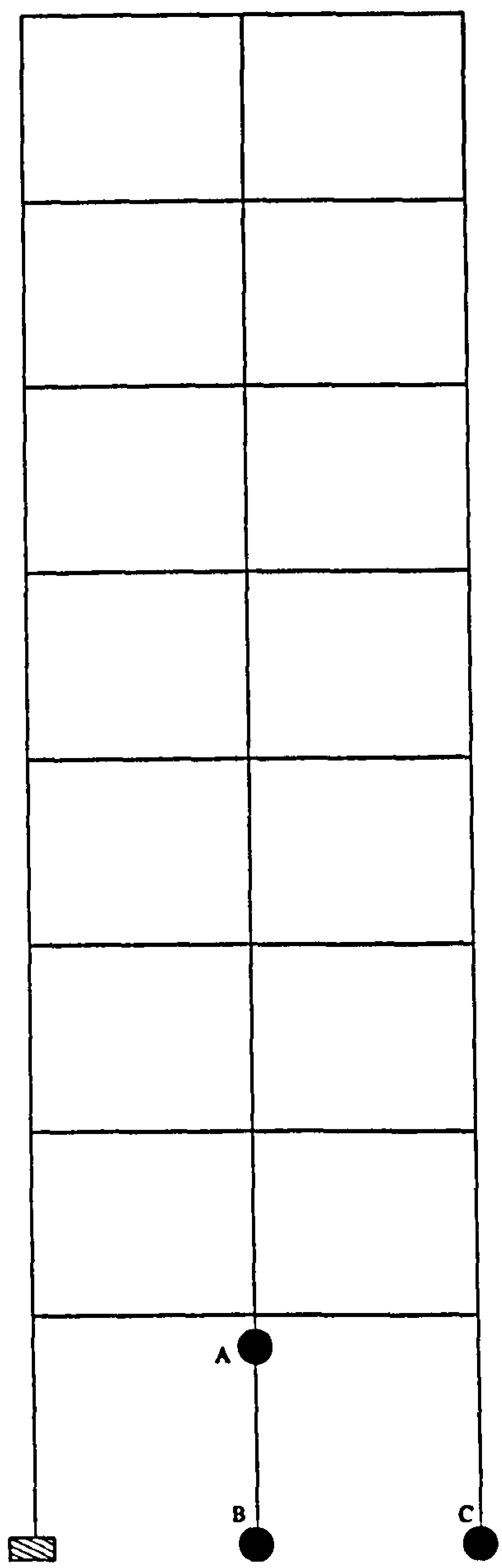


Frame 5
Load Case 3

Semi-Rigid Frame
(Section Designation II)



Rigid Frame
(Section Designation II)



Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.65	1.80
B	1.65	1.84
C	1.65	1.78
D	1.63	N/A

Key:

Semi-rigid connection

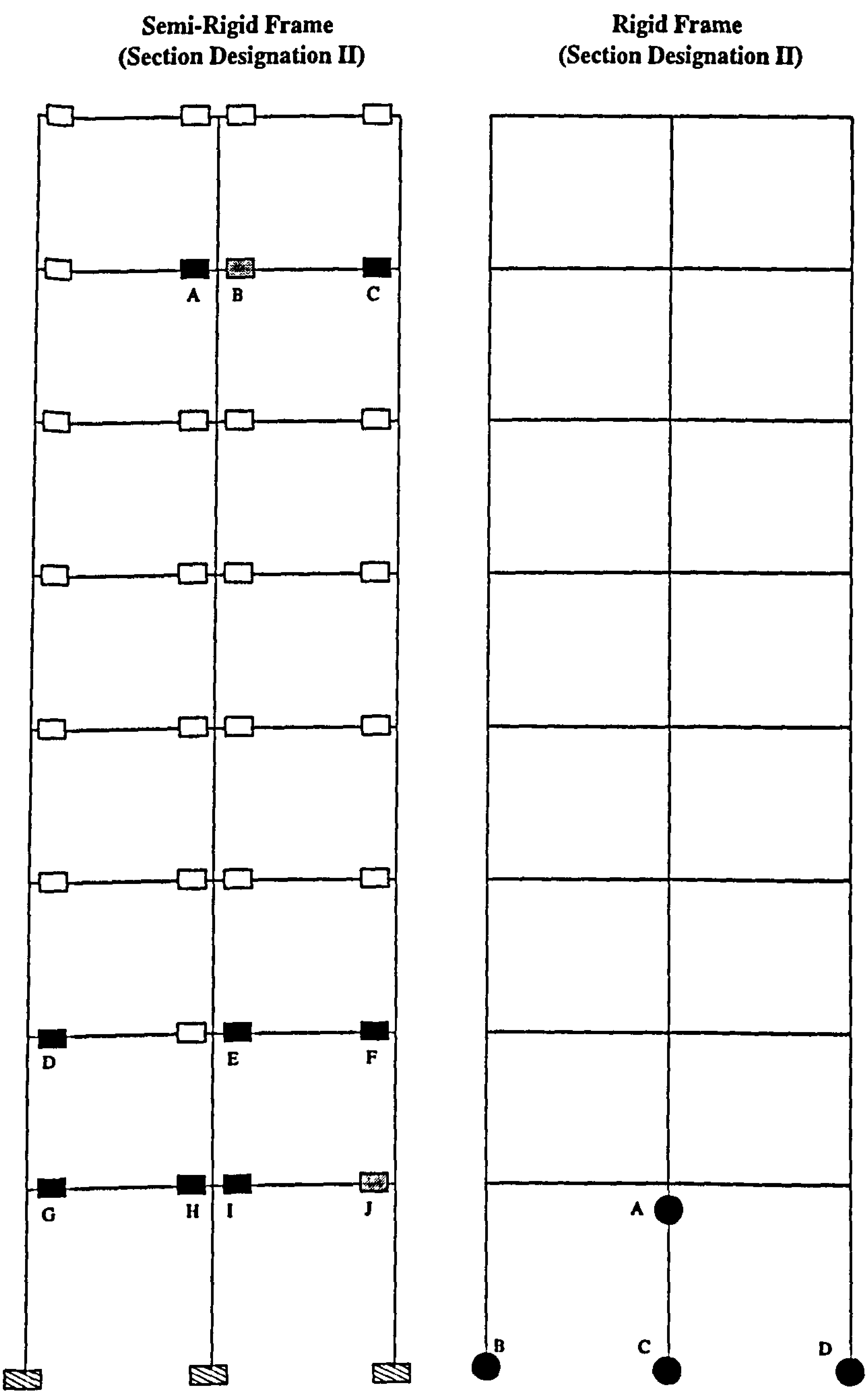
Plastification prior to ULS design load being attained

Plastification following the attainment of the design load

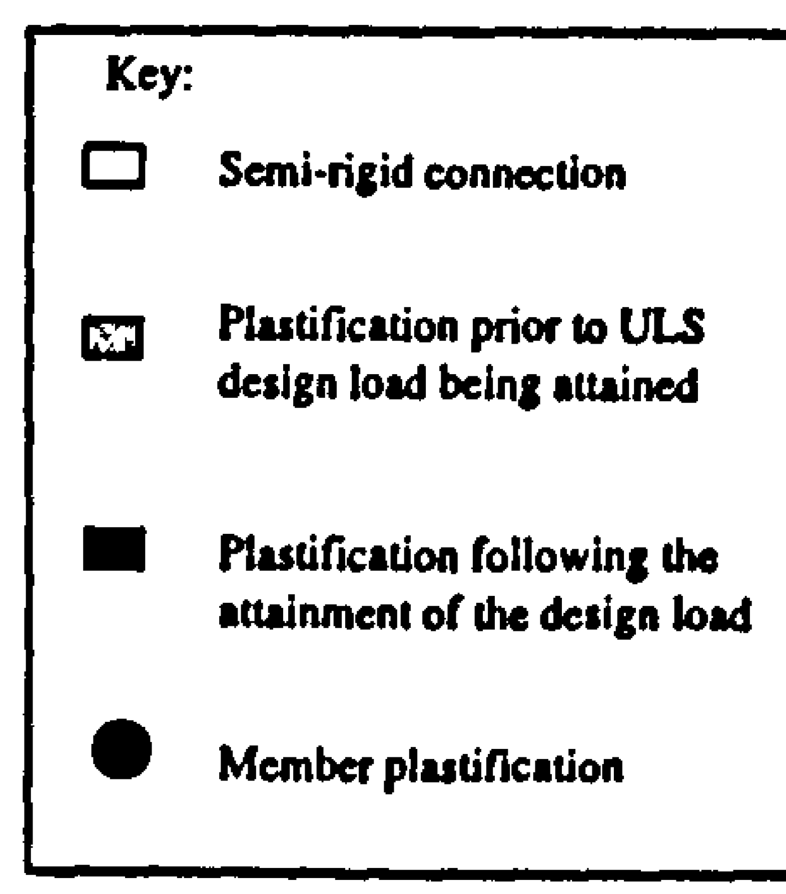
Member plastification

Frame 6
Load Case 1

Plastic-hinges in minor -axis "wind-moment" frames.

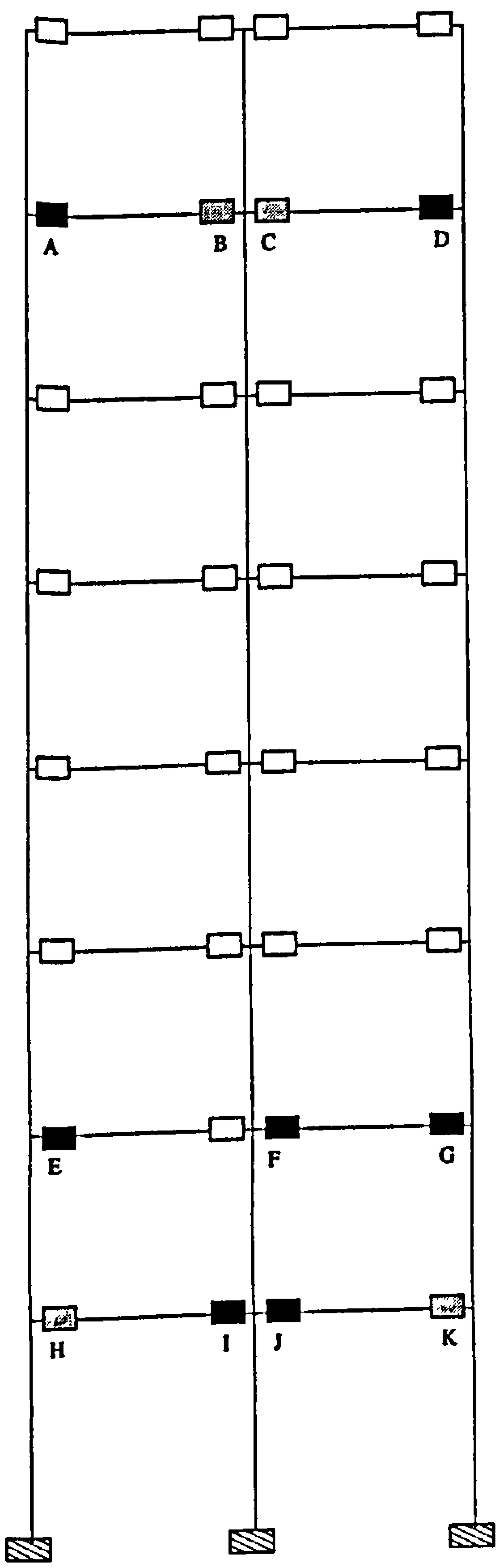


Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.03	1.60
B	< 1.00	1.53
C	1.03	1.57
D	1.03	1.59
E	1.03	N/A
F	1.03	N/A
G	1.03	N/A
H	1.03	N/A
I	1.03	N/A
J	< 1.00	N/A

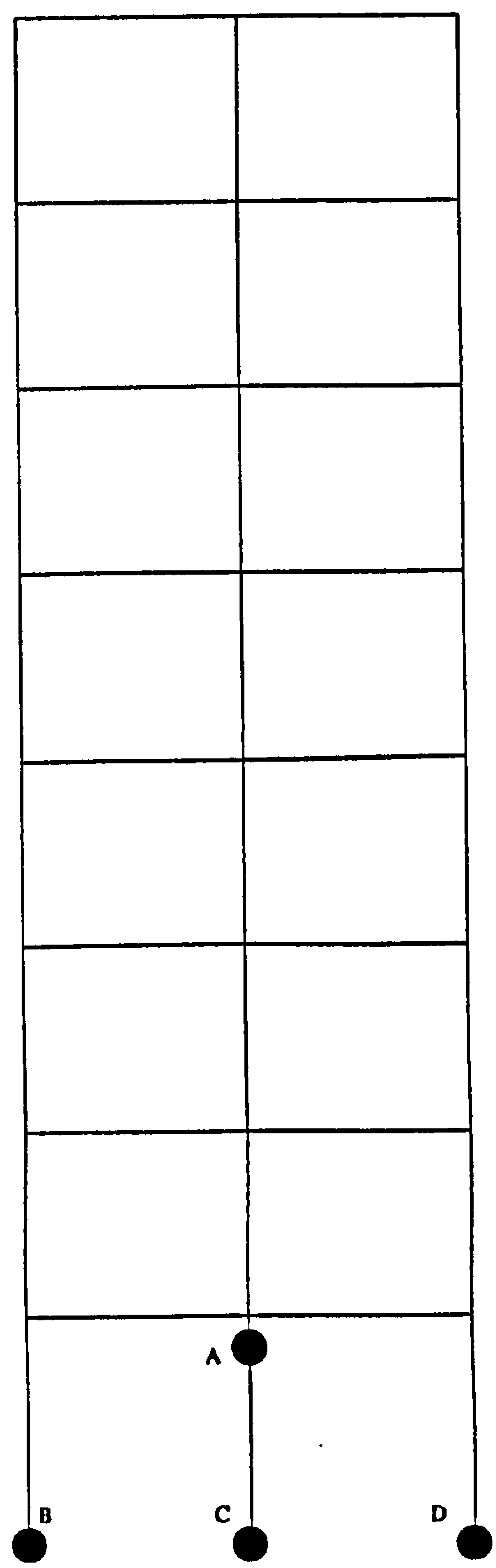


Frame 6
Load Case 2

Semi-Rigid Frame
(Section Designation II)



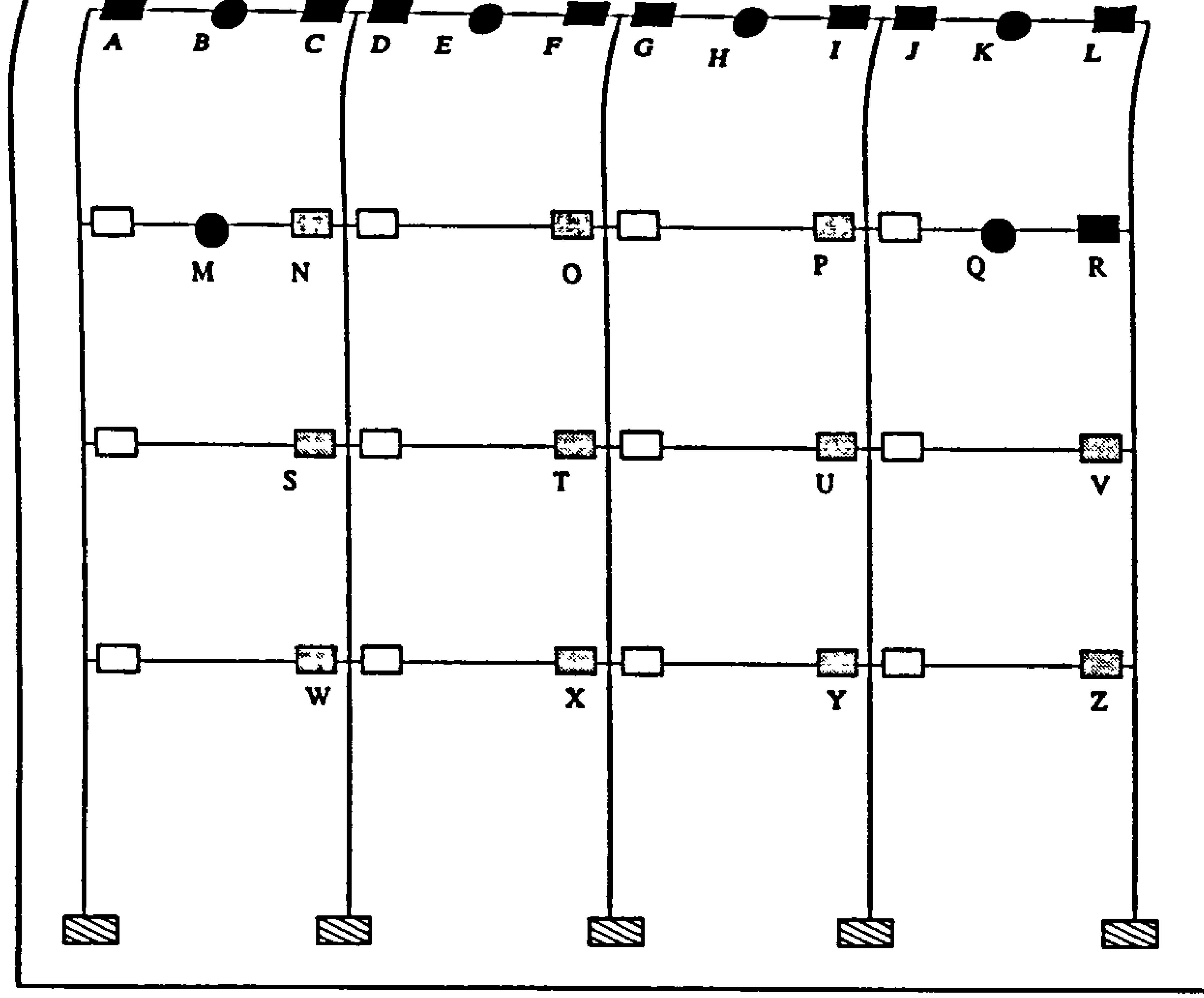
Rigid Frame
(Section Designation II)



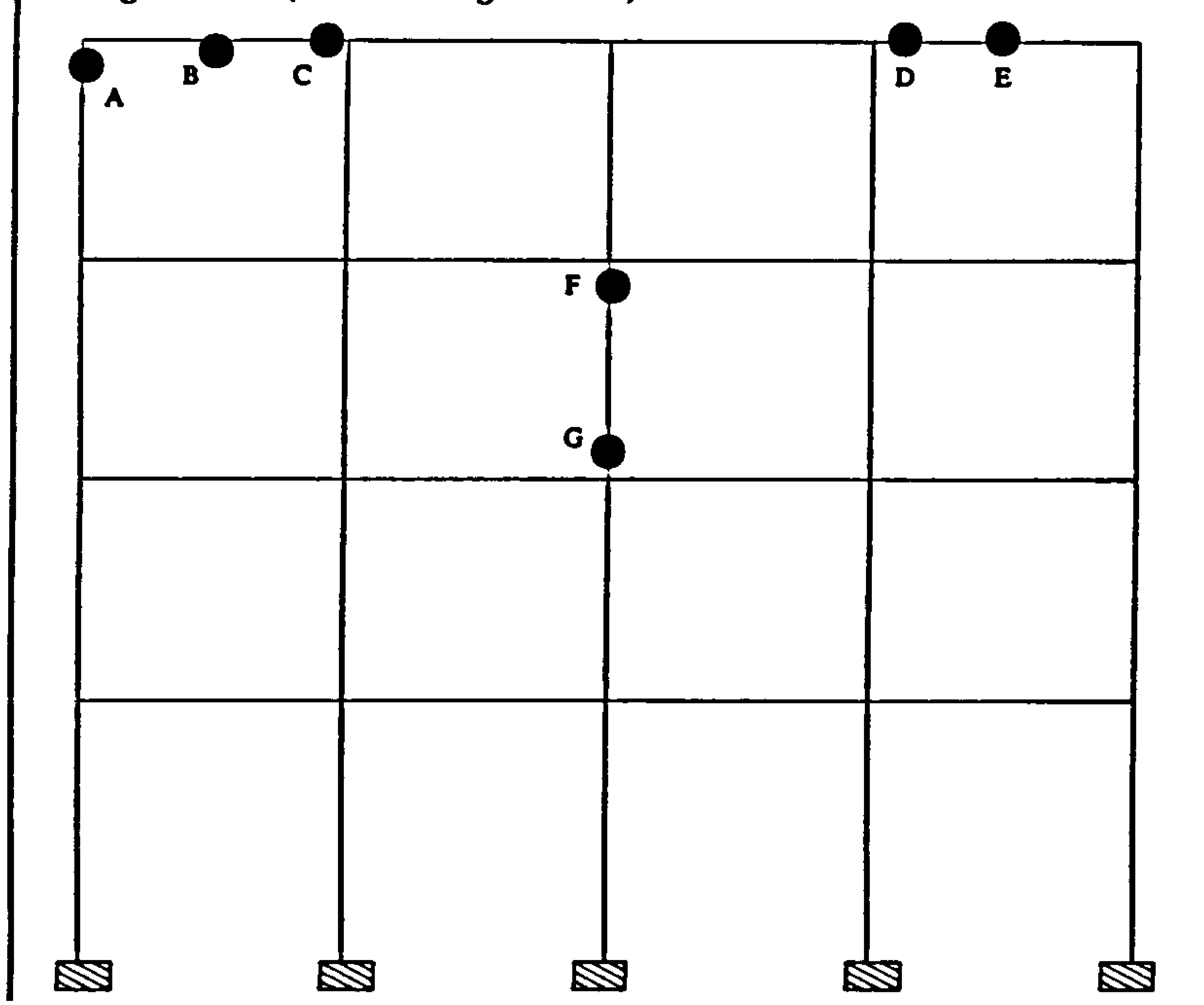
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.01	1.82
B	<1.00	1.81
C	<1.00	1.74
D	1.01	1.78
E	1.01	N/A
F	1.01	N/A
G	1.01	N/A
H	<1.00	N/A
I	1.01	N/A
J	1.01	N/A
K	<1.00	N/A

Key:
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 6
Load Case 3



Rigid Frame (Section Designation II)



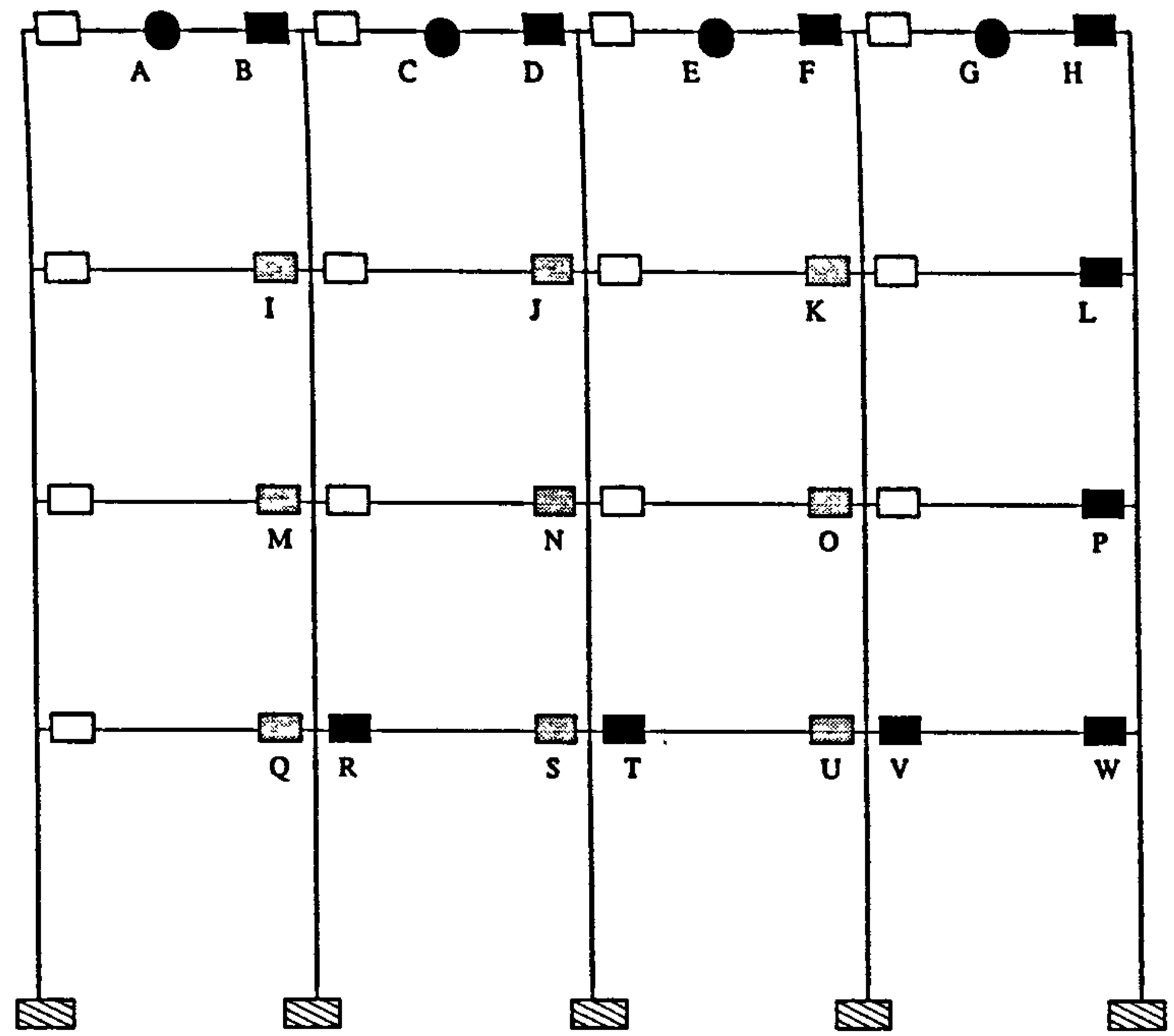
Location	Semi-rigid	Rigid
A	1.31	1.83
B	1.24	1.67
C	1.24	1.71
D	1.31	1.74
E	1.29	1.68
F	1.24	1.83
G	1.31	1.82
H	1.29	N/A
I	1.24	N/A
J	1.31	N/A
K	1.28	N/A
L	1.30	N/A
M	1.30	N/A
N	< 1.00	N/A
O	< 1.00	N/A
P	< 1.00	N/A
Q	1.31	N/A
R	1.24	N/A
S	< 1.00	N/A
T	< 1.00	N/A
U	< 1.00	N/A
V	< 1.00	N/A
W	< 1.00	N/A
X	< 1.00	N/A
Y	< 1.00	N/A
Z	< 1.00	N/A

Key:

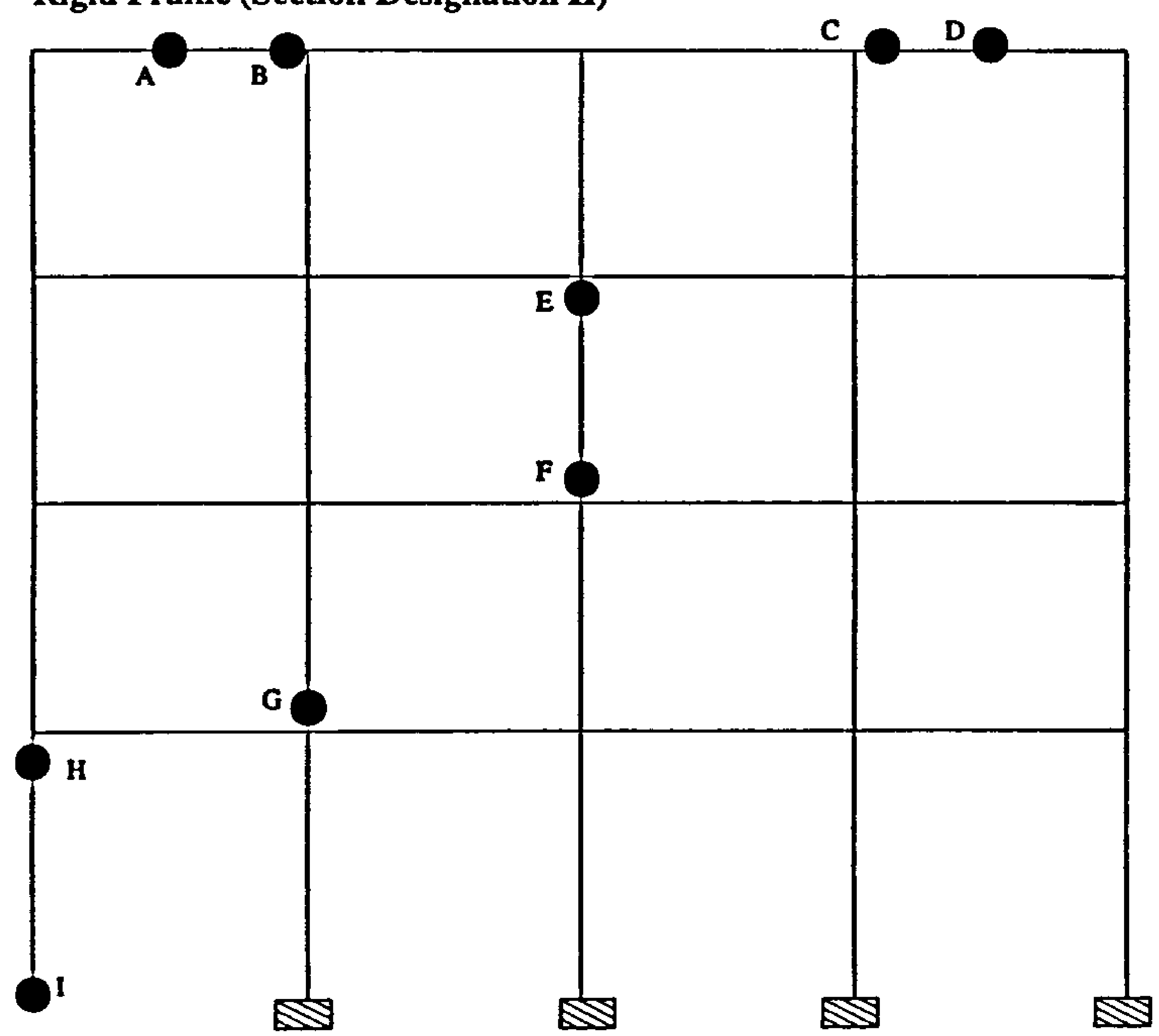
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 7
Load Case 1

Semi-Rigid Frame (Section Designation III)



Rigid Frame (Section Designation II)

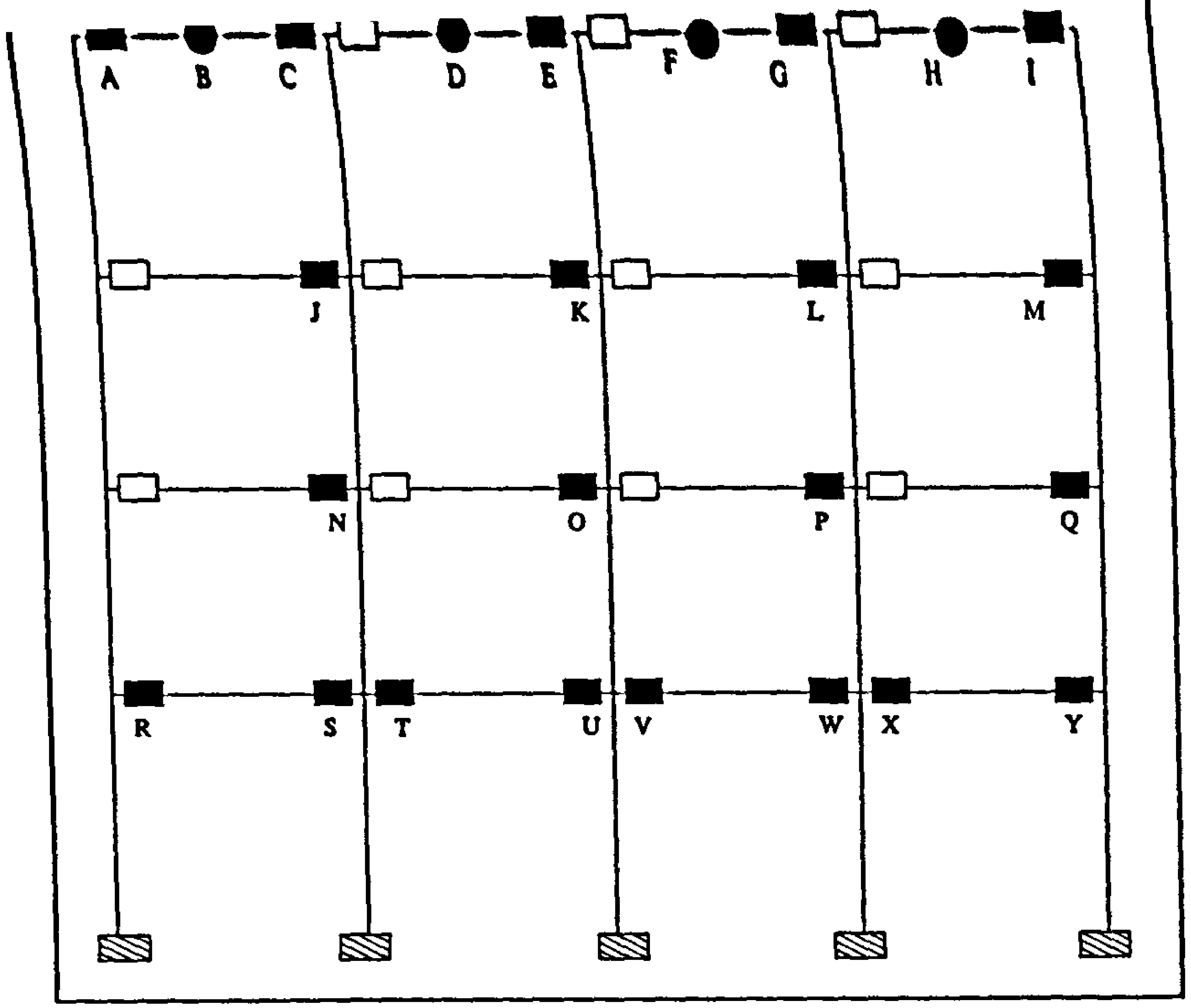


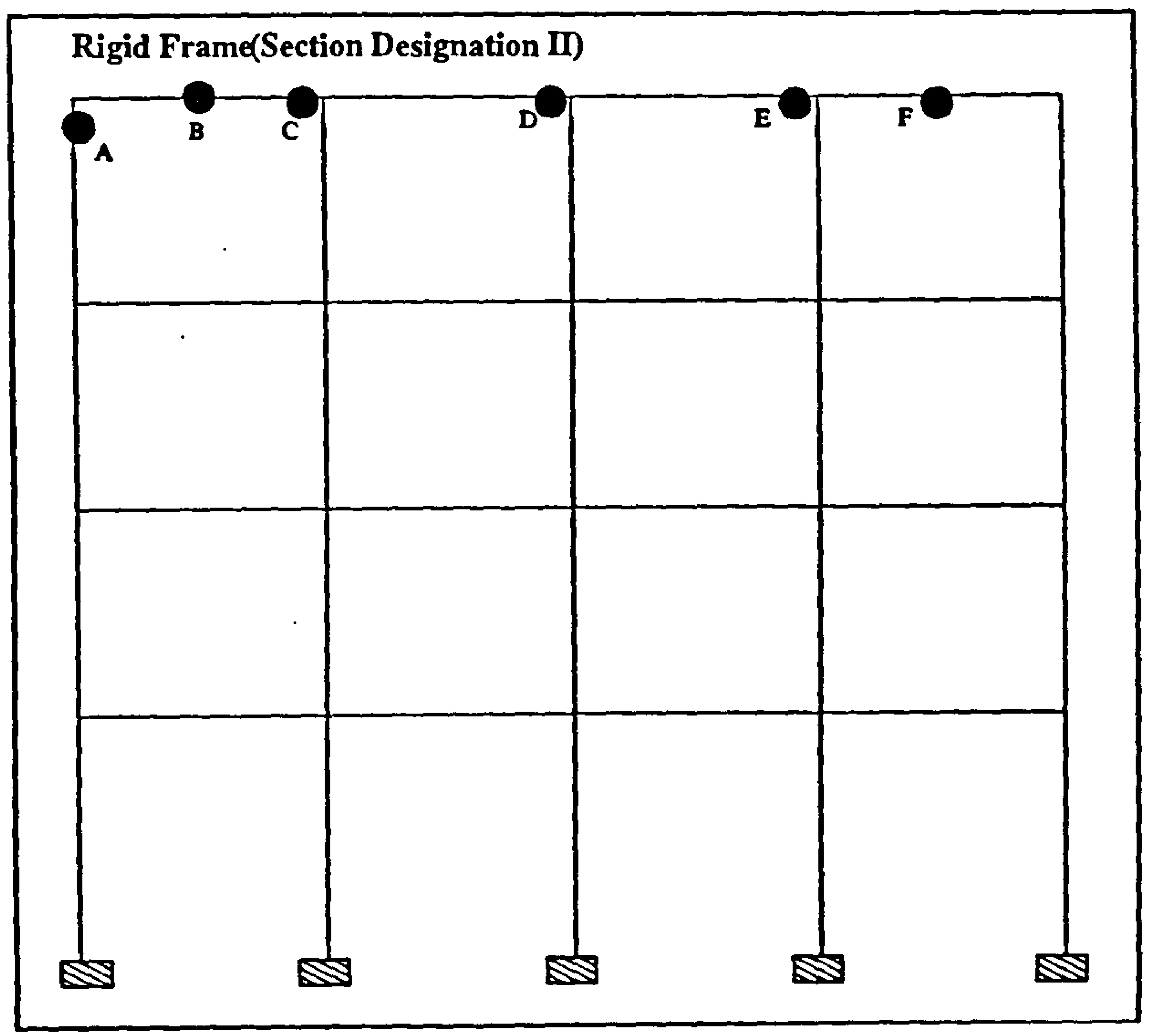
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.48	2.02
B	1.48	2.11
C	1.53	2.04
D	1.48	2.13
E	1.53	2.11
F	1.48	2.15
G	1.53	2.16
H	1.53	2.14
I	< 1.00	2.14
J	< 1.00	N/A
K	< 1.00	N/A
L	1.48	N/A
M	< 1.00	N/A
N	< 1.00	N/A
O	< 1.00	N/A
P	1.48	N/A
Q	< 1.00	N/A
R	1.53	N/A
S	< 1.00	N/A
T	1.53	N/A
U	< 1.00	N/A
V	1.53	N/A
W	1.48	N/A

Key:

- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

**Frame 7
Load Case 2**





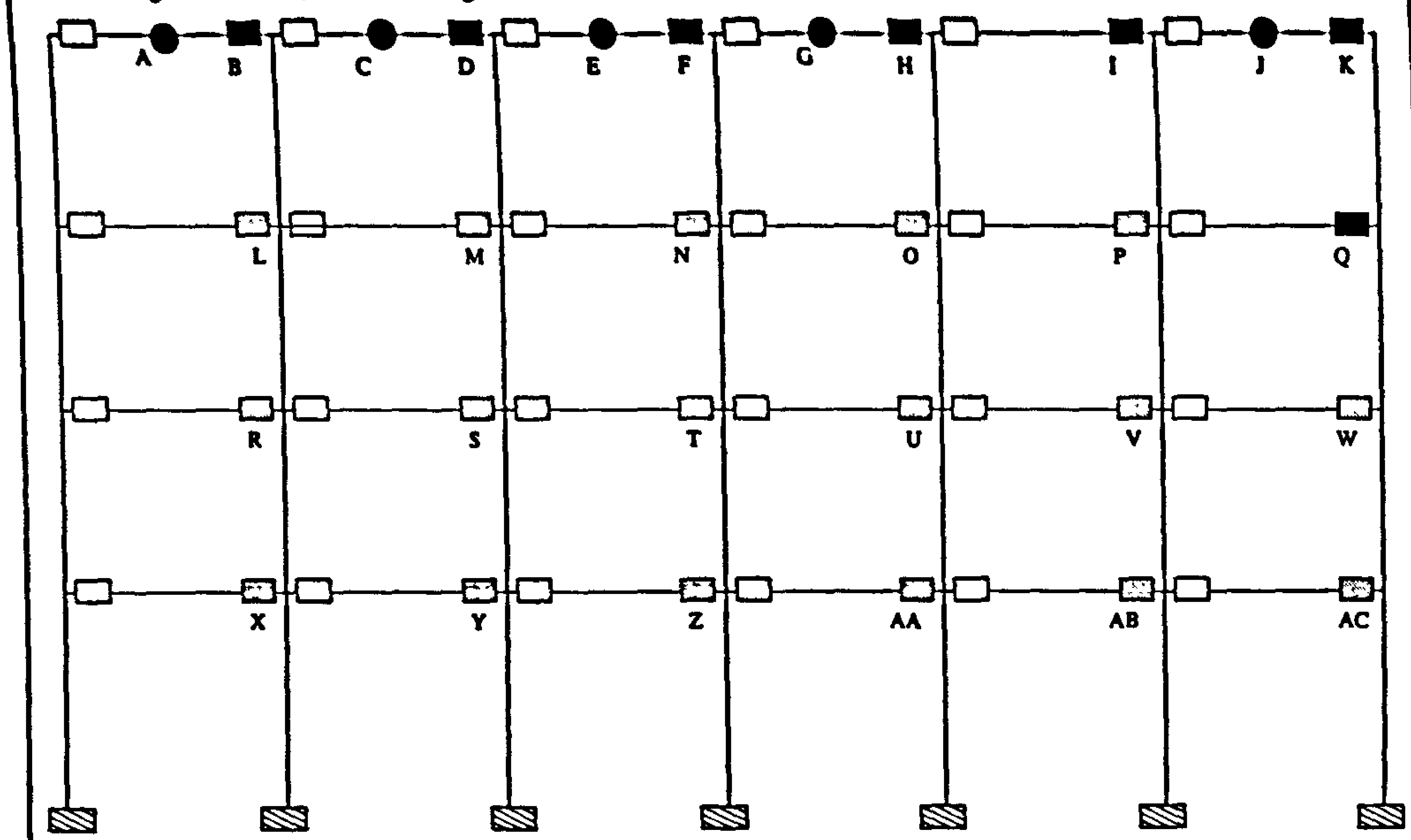
Loaction	Semi-rigid	Rigid
A	1.89	2.64
B	1.77	2.39
C	1.77	2.38
D	1.84	2.62
E	1.77	2.47
F	1.84	2.45
G	1.77	N/A
H	1.84	N/A
I	1.84	N/A
J	1.77	N/A
K	1.77	N/A
L	1.77	N/A
M	1.77	N/A
N	1.77	N/A
O	1.77	N/A
P	1.77	N/A
Q	1.77	N/A
R	1.89	N/A
S	1.77	N/A
T	1.89	N/A
U	1.77	N/A
V	1.89	N/A
W	1.77	N/A
X	1.89	N/A
Y	1.77	N/A

Key:

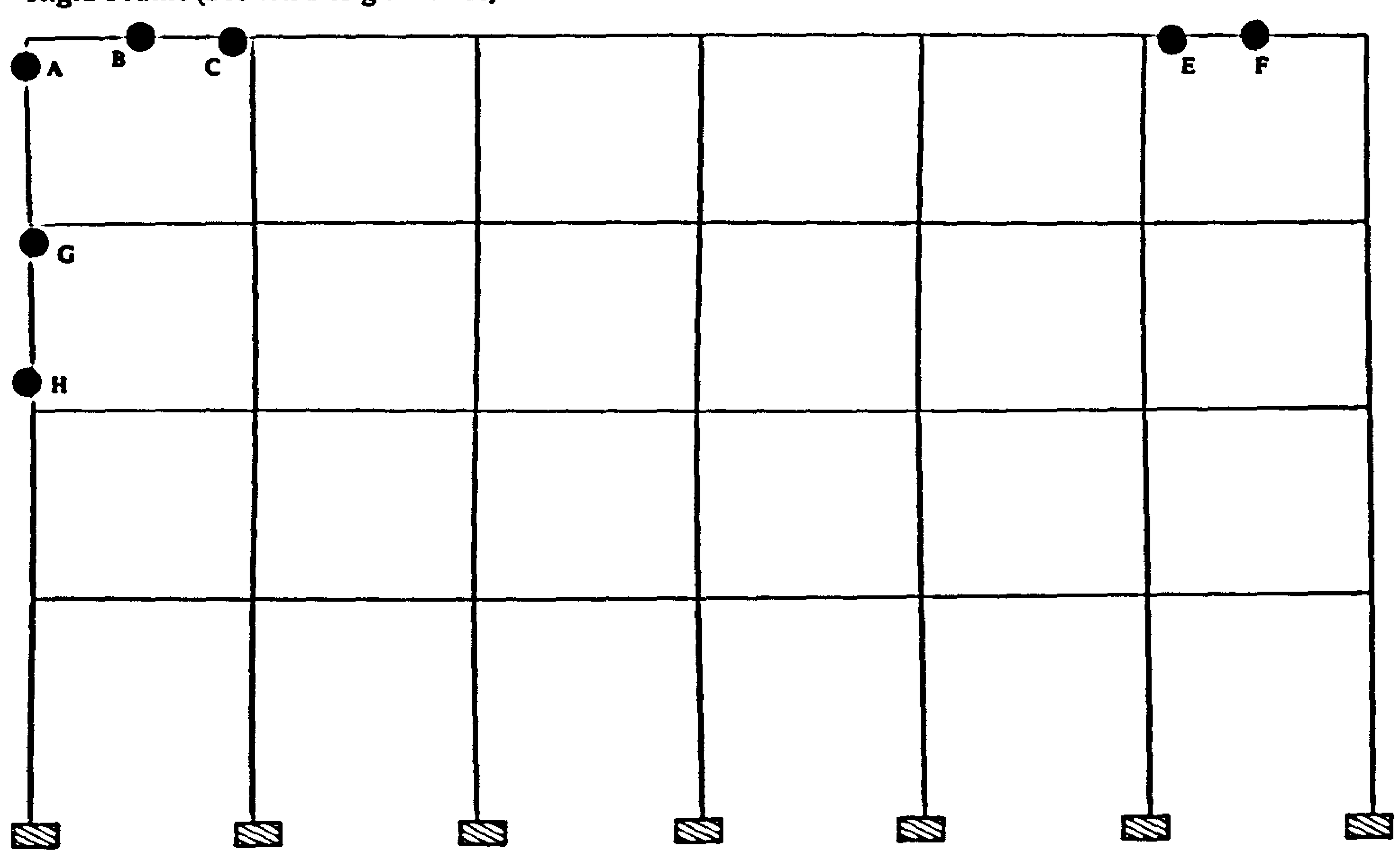
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 7
Load Case 3

Semi-Rigid Frame (Section Designation II)



Rigid Frame (Section Designation II)

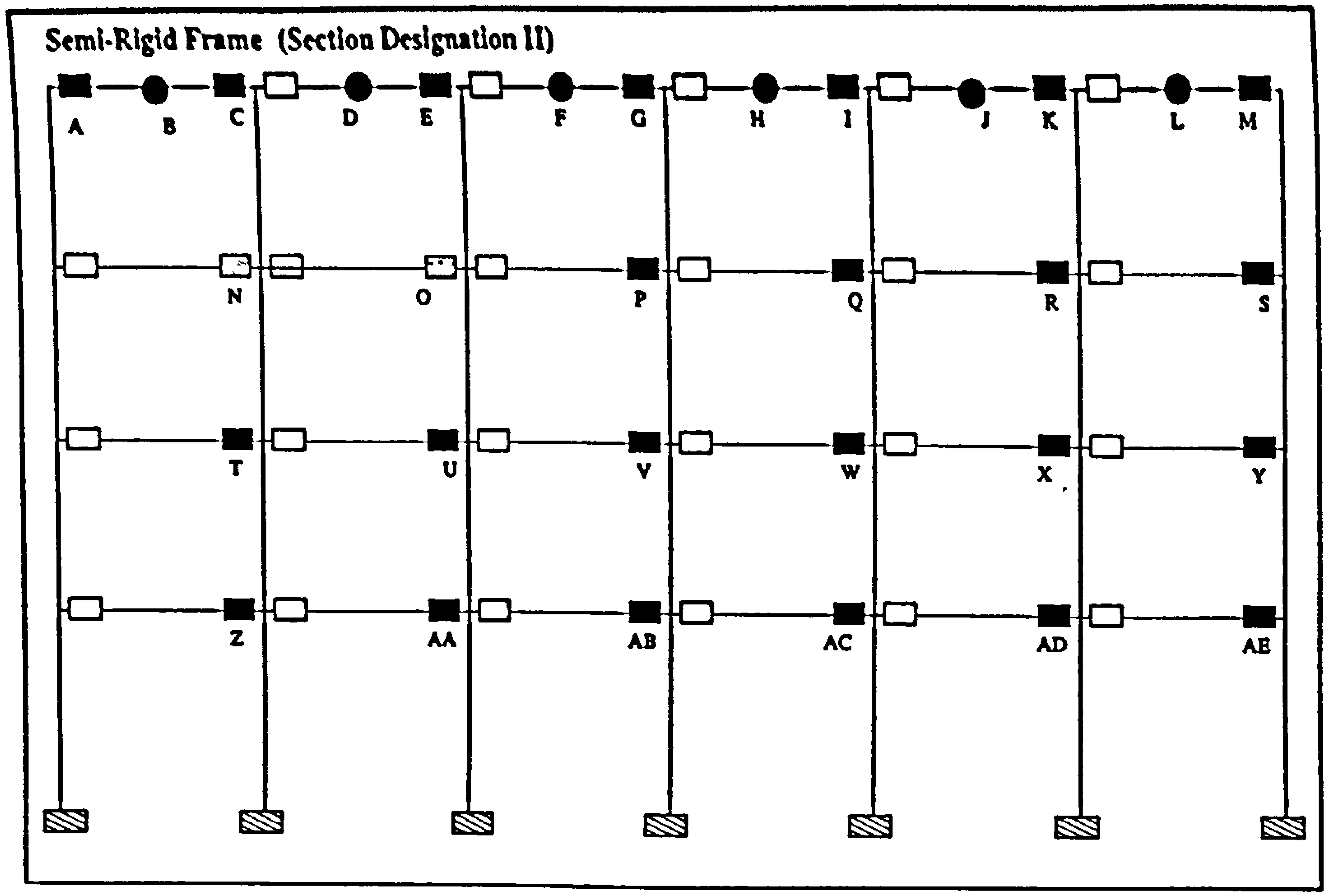


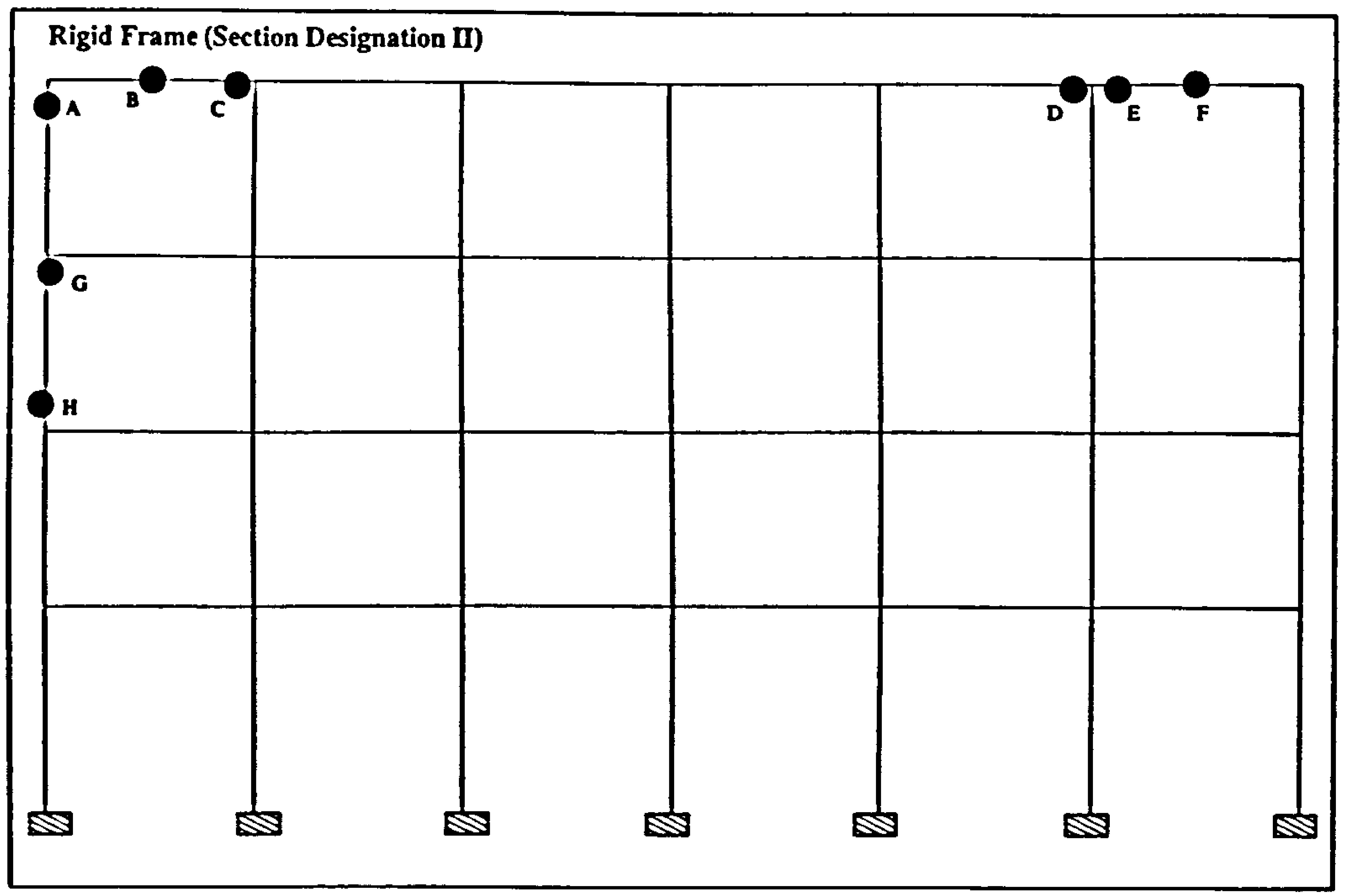
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.24	1.83
B	1.24	1.69
C	1.29	1.70
D	1.24	1.77
E	1.29	1.70
F	1.24	1.73
G	1.29	1.74
H	1.24	N/A
I	1.24	N/A
J	1.28	N/A
K	1.28	N/A
L	< 1.00	N/A
M	< 1.00	N/A
N	< 1.00	N/A
O	< 1.00	N/A
P	< 1.00	N/A
Q	1.24	N/A
R	< 1.00	N/A
S	< 1.00	N/A
T	< 1.00	N/A
U	< 1.00	N/A
V	< 1.00	N/A
W	< 1.00	N/A
X	< 1.00	N/A
Y	< 1.00	N/A
Z	< 1.00	N/A
AA	< 1.00	N/A
AB	< 1.00	N/A
AC	< 1.00	N/A

Key:

- □ Semi-rigid connection
- □ Plastification prior to ULS design load being attained
- ■ Plastification following the attainment of the design load
- ● Member plastification

Frame 8
Load Case 1





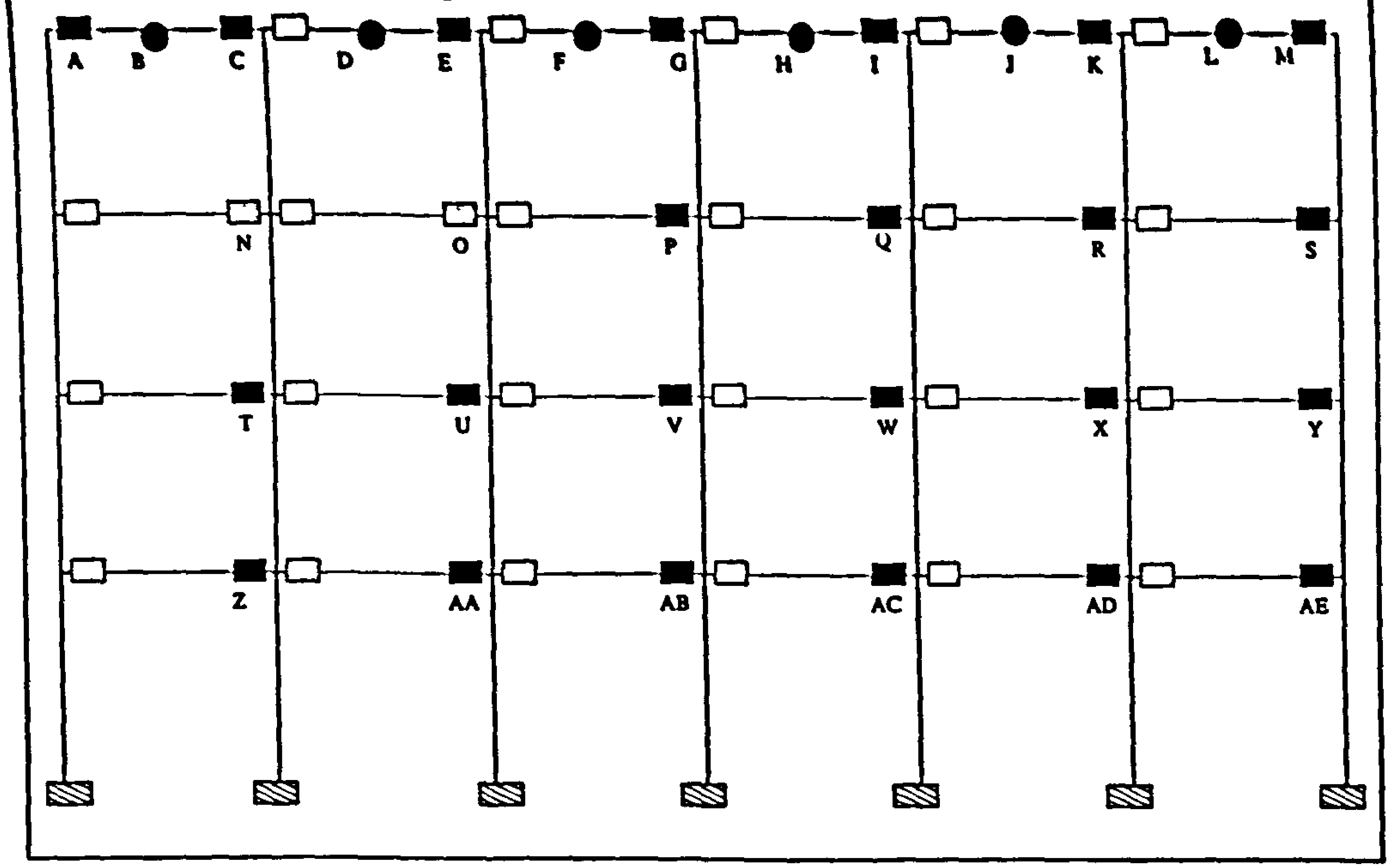
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.59	2.21
B	1.49	2.04
C	1.49	2.04
D	1.55	2.18
E	1.49	2.18
F	1.55	2.07
G	1.49	2.17
H	1.55	2.18
I	1.49	N/A
J	1.55	N/A
K	1.49	N/A
L	1.54	N/A
M	1.54	N/A
N	< 1.00	N/A
O	< 1.00	N/A
P	1.49	N/A
Q	1.49	N/A
R	1.49	N/A
S	1.49	N/A
T	1.49	N/A
U	1.49	N/A
V	1.49	N/A
W	1.49	N/A
X	1.49	N/A
Y	1.49	N/A
Z	1.49	N/A
AA	1.49	N/A
AB	1.49	N/A
AC	1.49	N/A
AD	1.49	N/A
AE	1.49	N/A

Key:

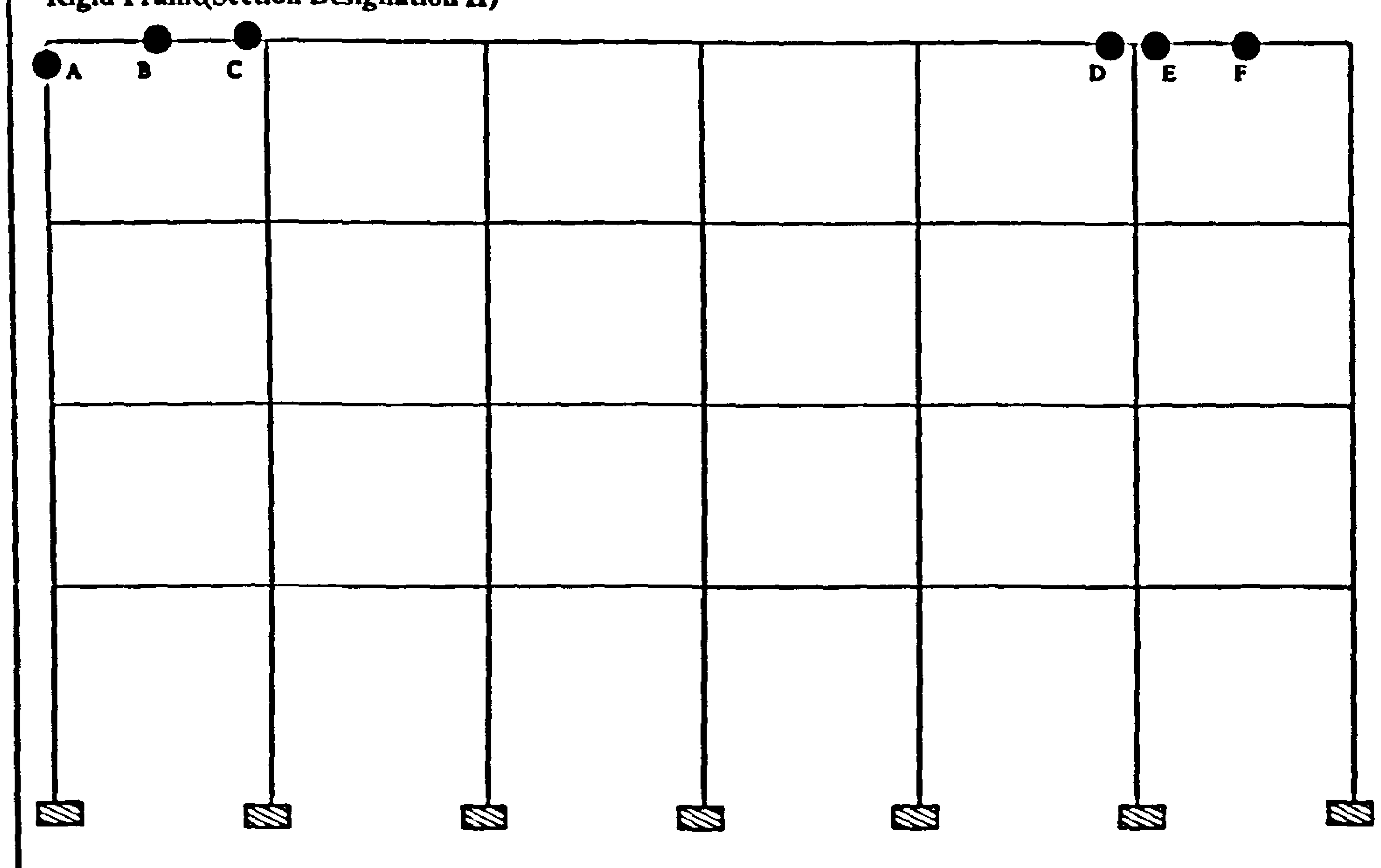
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 8
Load Case 2

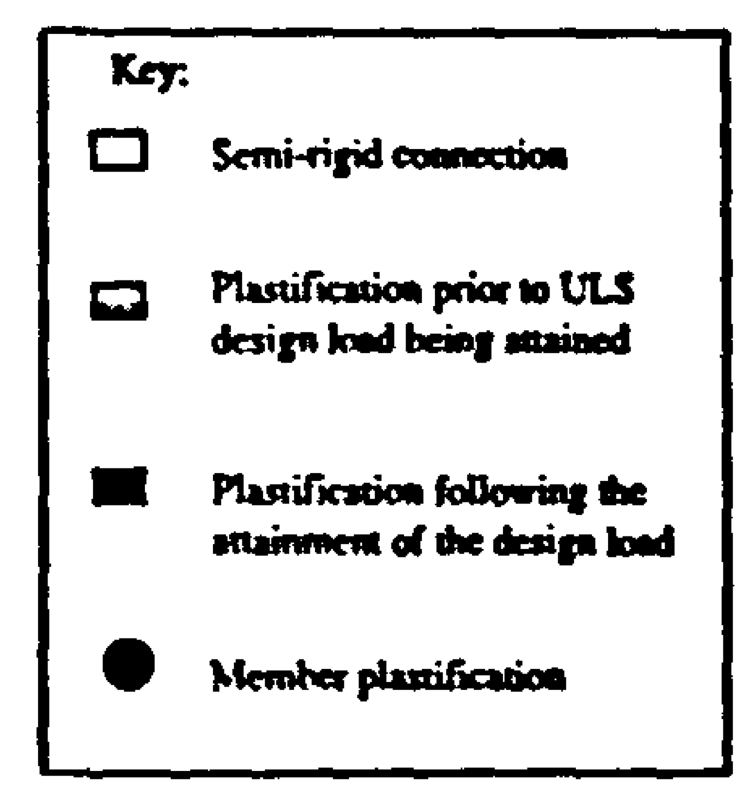
Semi-Rigid Frame (Section Designation II)



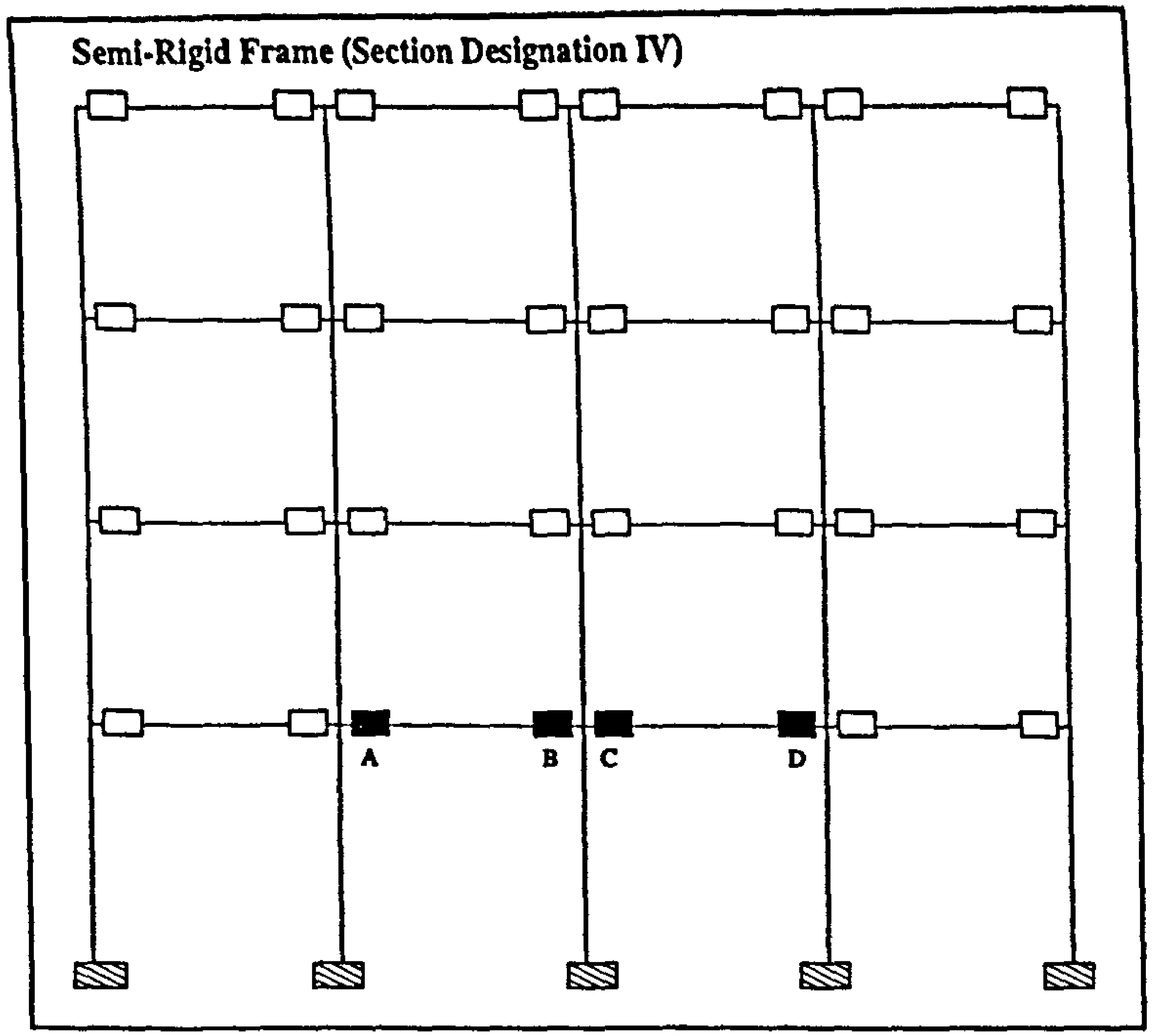
Rigid Frame(Section Designation II)

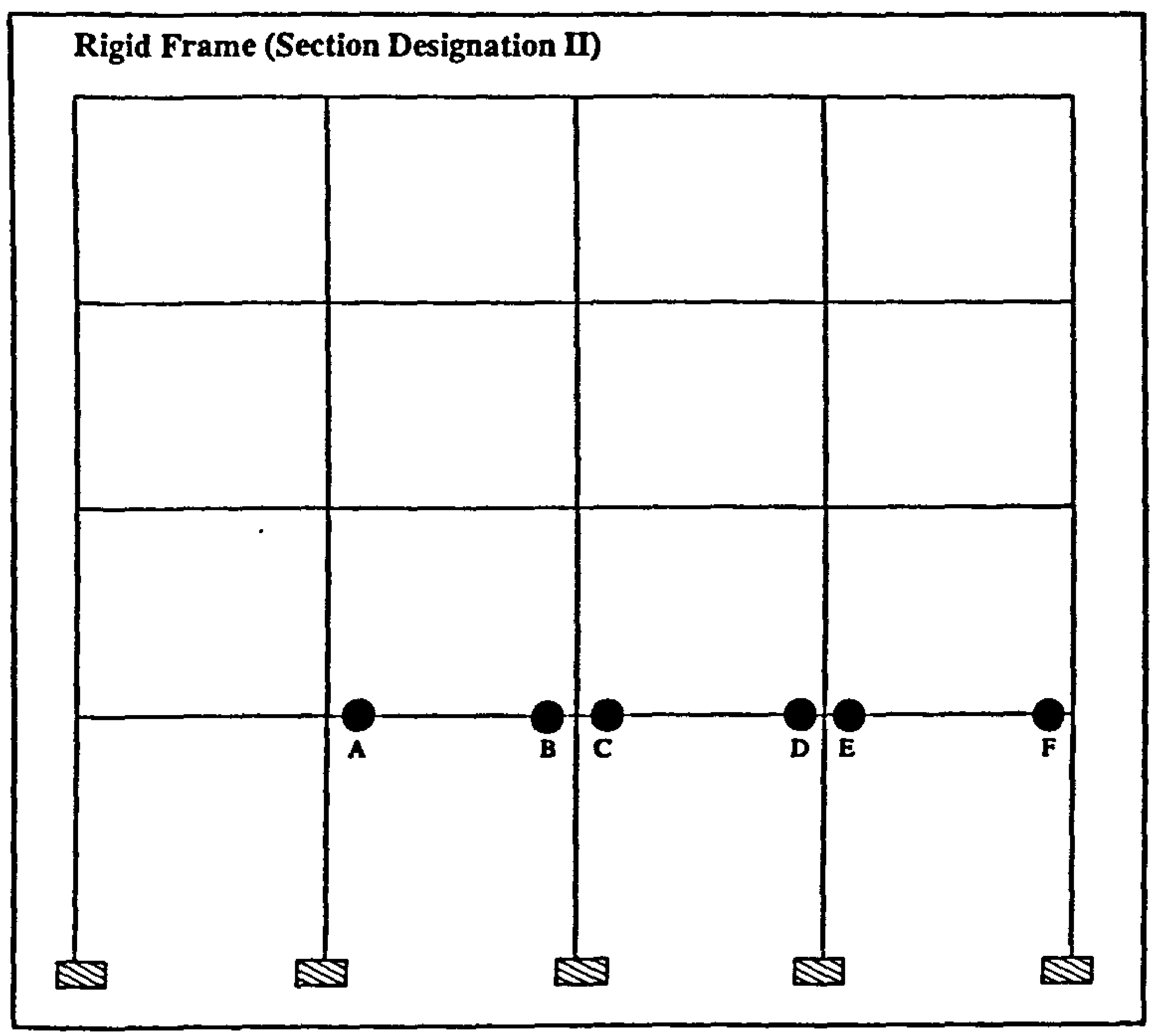


Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.90	2.65
B	1.79	2.41
C	1.79	2.36
D	1.87	2.55
E	1.79	2.62
F	1.87	2.48
G	1.79	N/A
H	1.87	N/A
I	1.79	N/A
J	1.87	N/A
K	1.79	N/A
L	1.87	N/A
M	1.87	N/A
N	1.79	N/A
O	1.79	N/A
P	1.79	N/A
Q	1.79	N/A
R	1.79	N/A
S	1.79	N/A
T	1.79	N/A
U	1.79	N/A
V	1.79	N/A
W	1.79	N/A
X	1.87	N/A
Y	1.79	N/A
Z	1.79	N/A
AA	1.79	N/A
AB	1.79	N/A
AC	1.79	N/A
AD	1.79	N/A
AE	1.79	N/A



Frame 8
Load Case 3

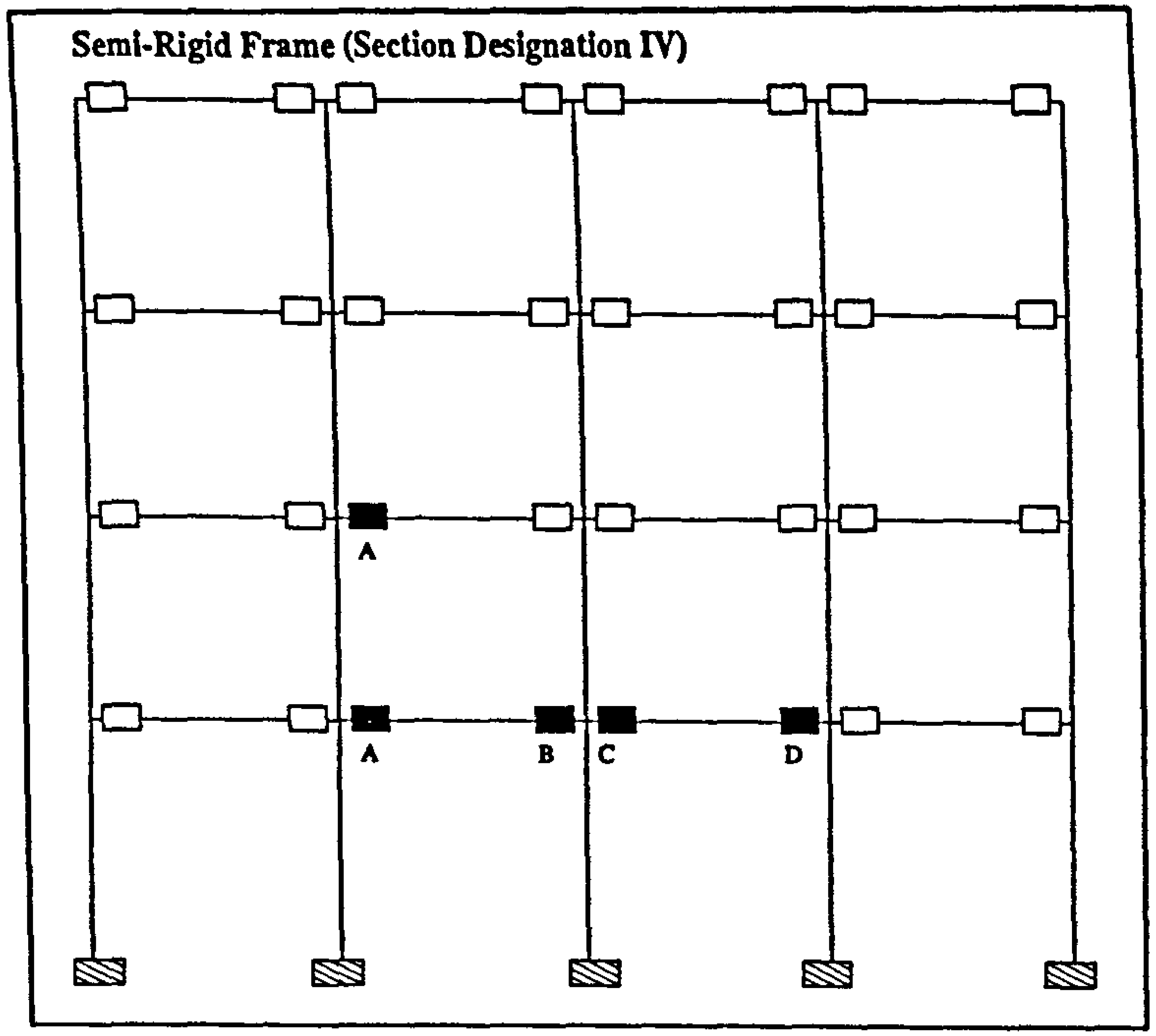


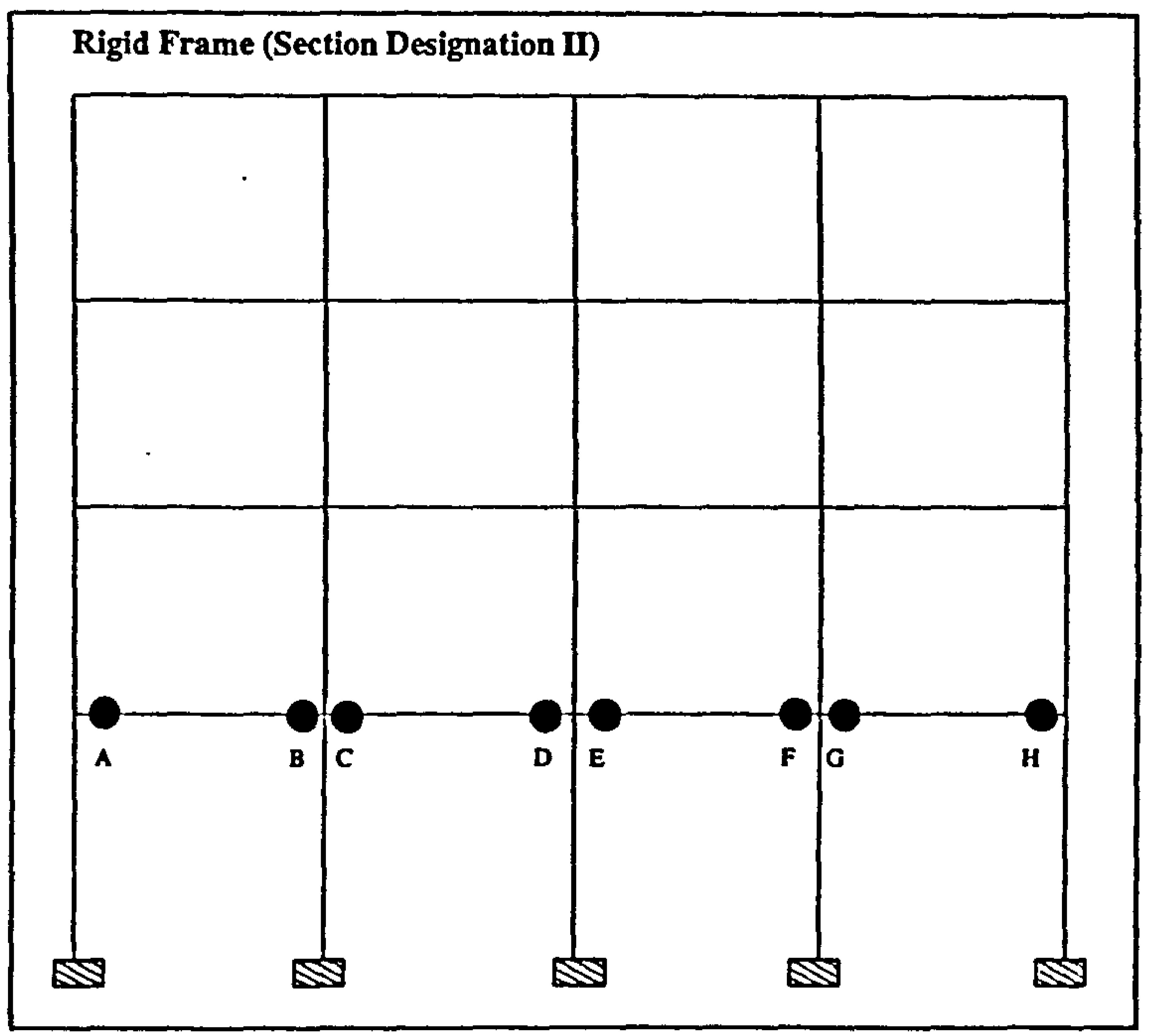


Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.37	1.38
B	1.37	1.38
C	1.37	1.38
D	1.37	1.38
E	N/A	1.38
F	N/A	1.38

Key:
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 9
Load Case 1

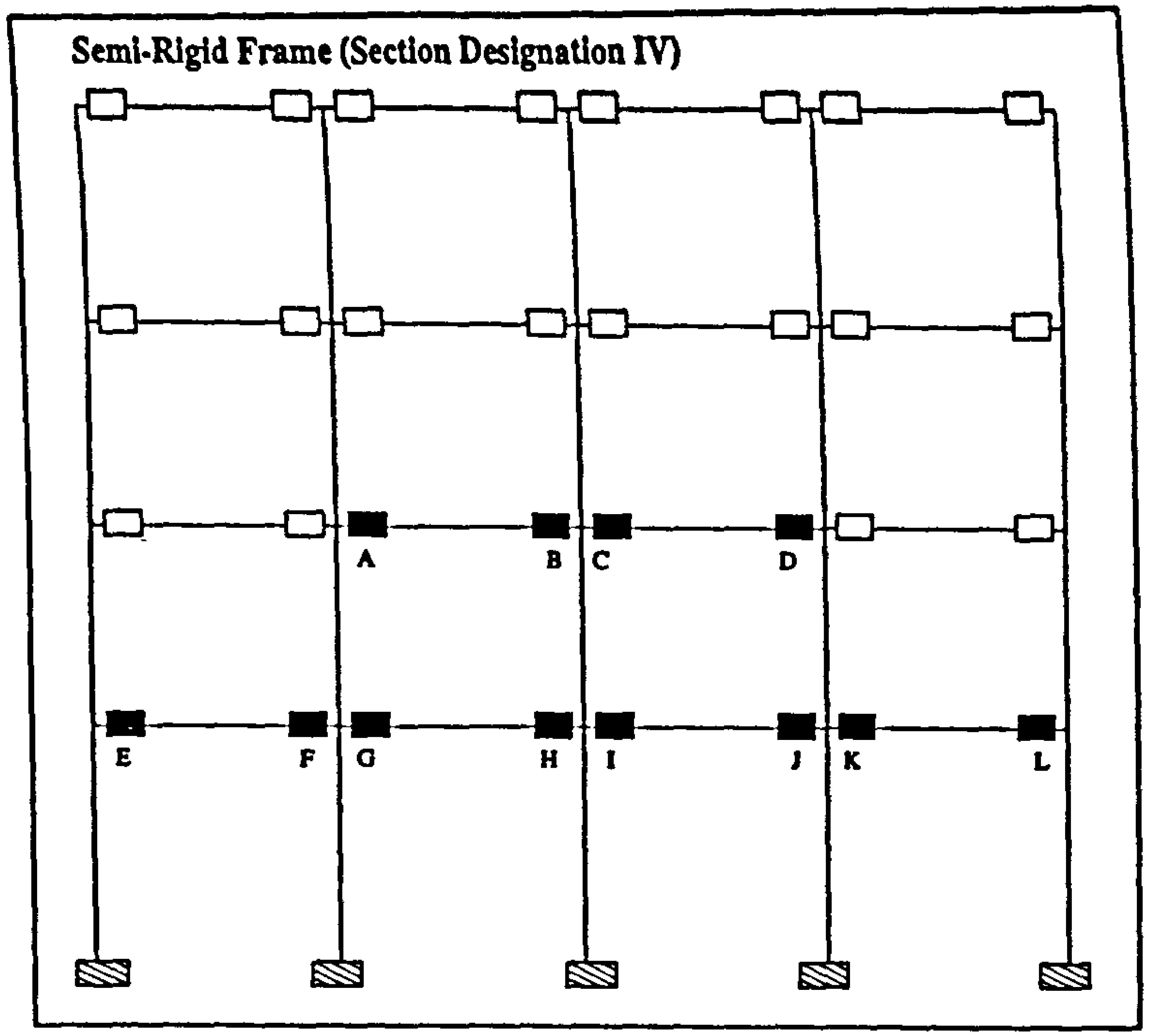


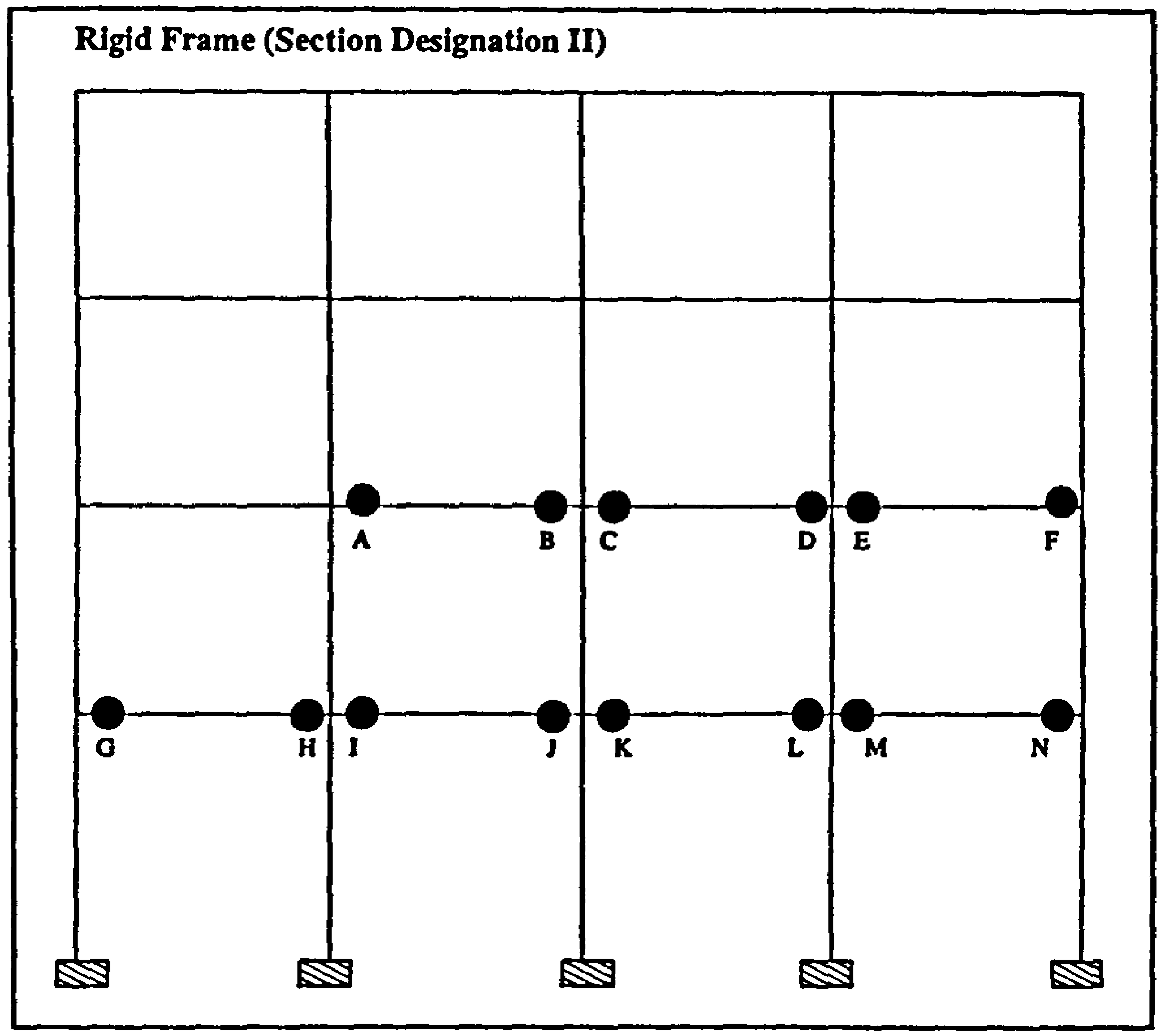


Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.36	1.46
B	1.36	1.46
C	1.36	1.45
D	1.36	1.45
E	1.36	1.45
F	N/A	1.45
G	N/A	1.46
H	N/A	1.46

Key:
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 9
Load Case 2

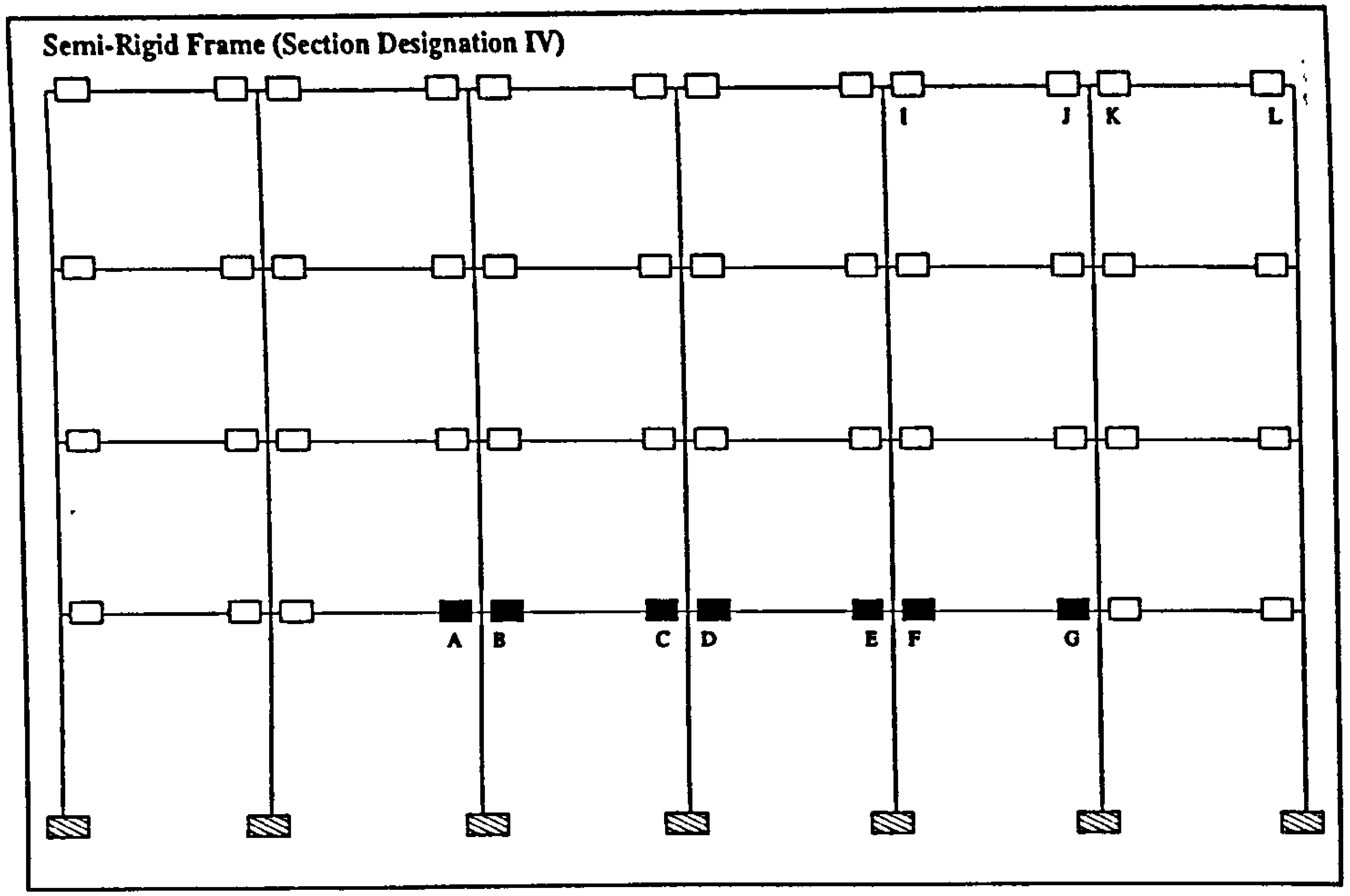


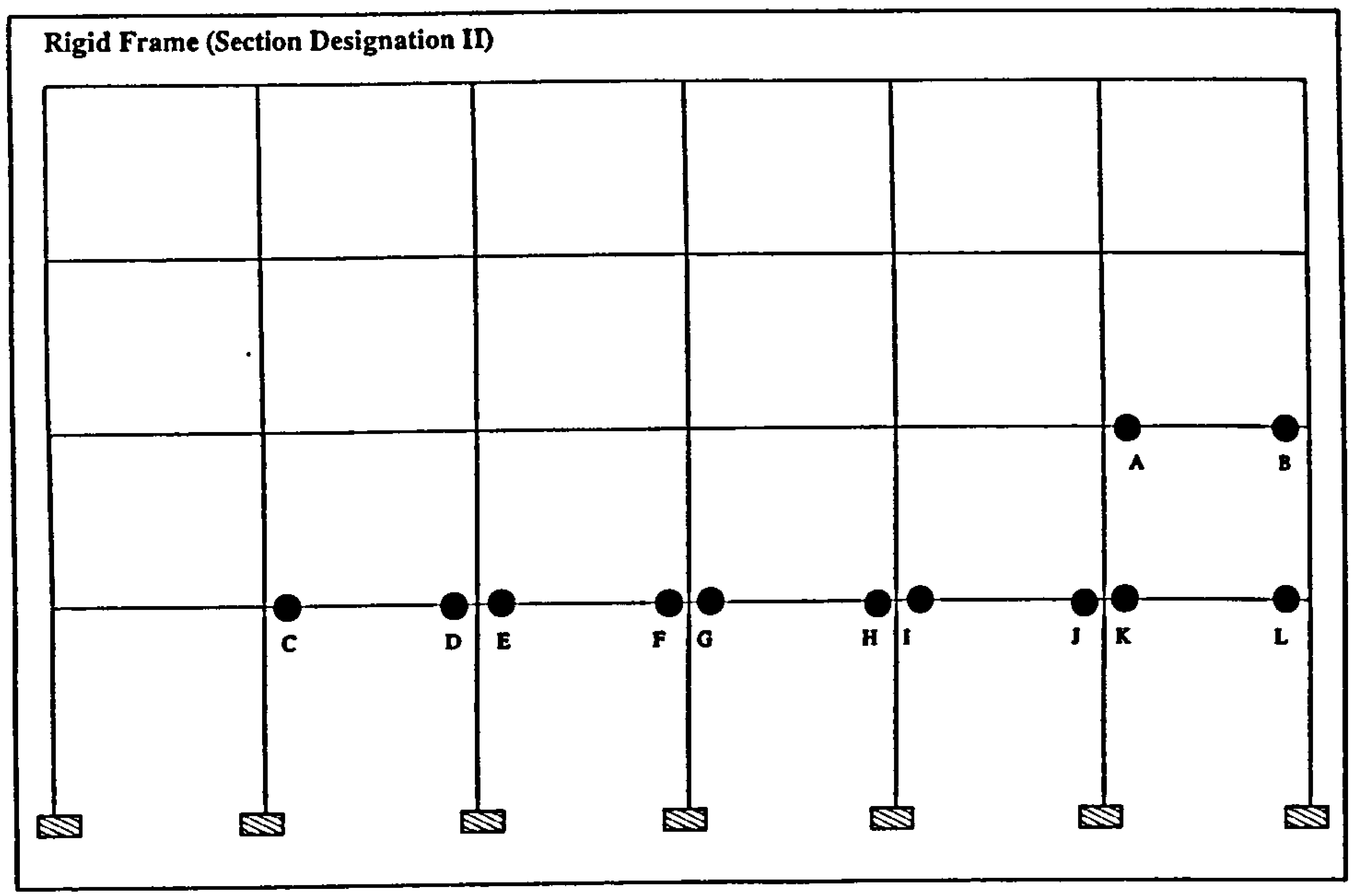


Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.58	1.74
B	1.58	1.74
C	1.58	1.74
D	1.58	1.74
E	1.58	1.74
F	1.58	1.74
G	1.58	1.73
H	1.58	1.73
I	1.58	1.72
J	1.58	1.72
K	1.58	1.73
L	1.58	1.73
M	N/A	1.72
N	N/A	1.72

Key:
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 9
Load Case 3

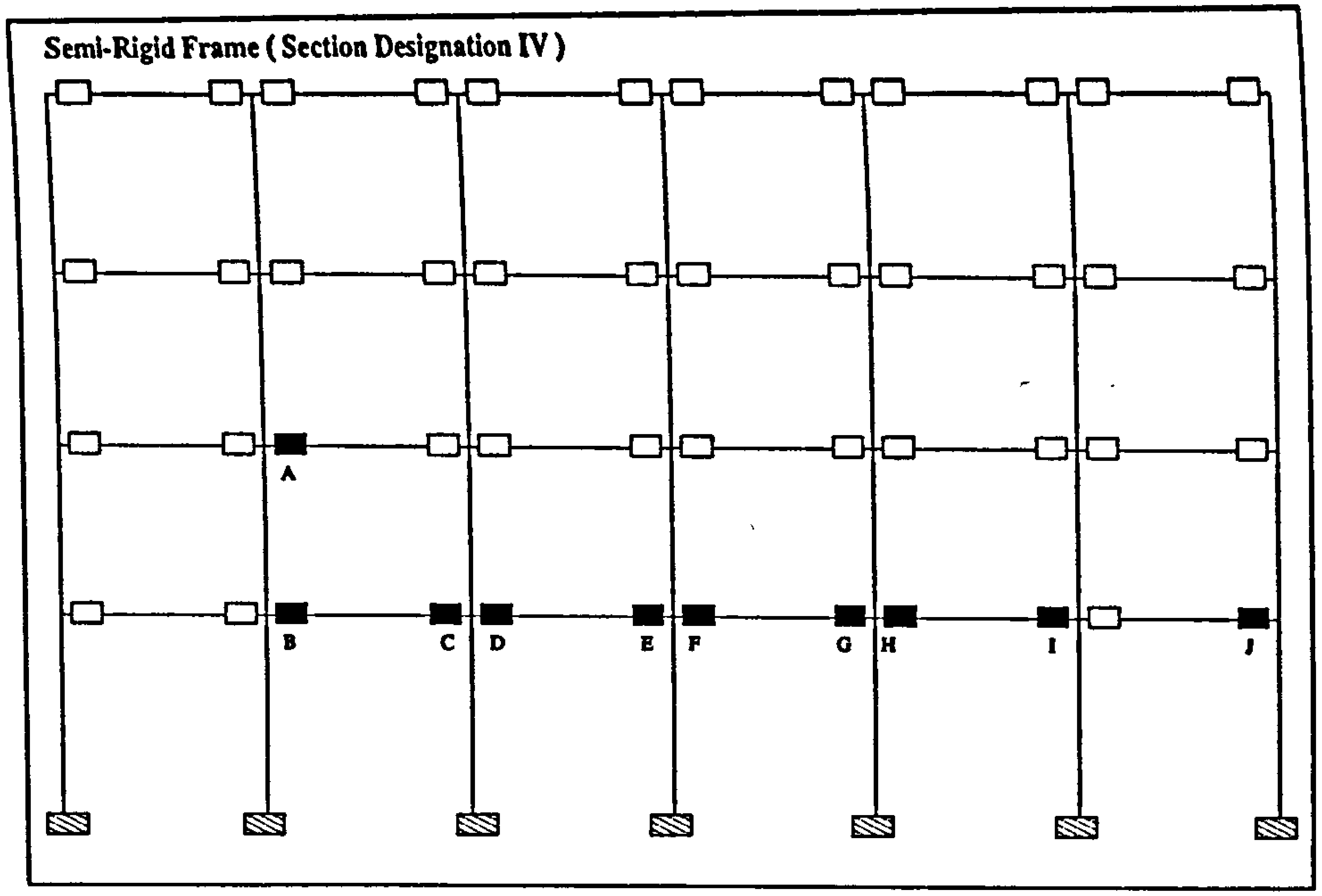


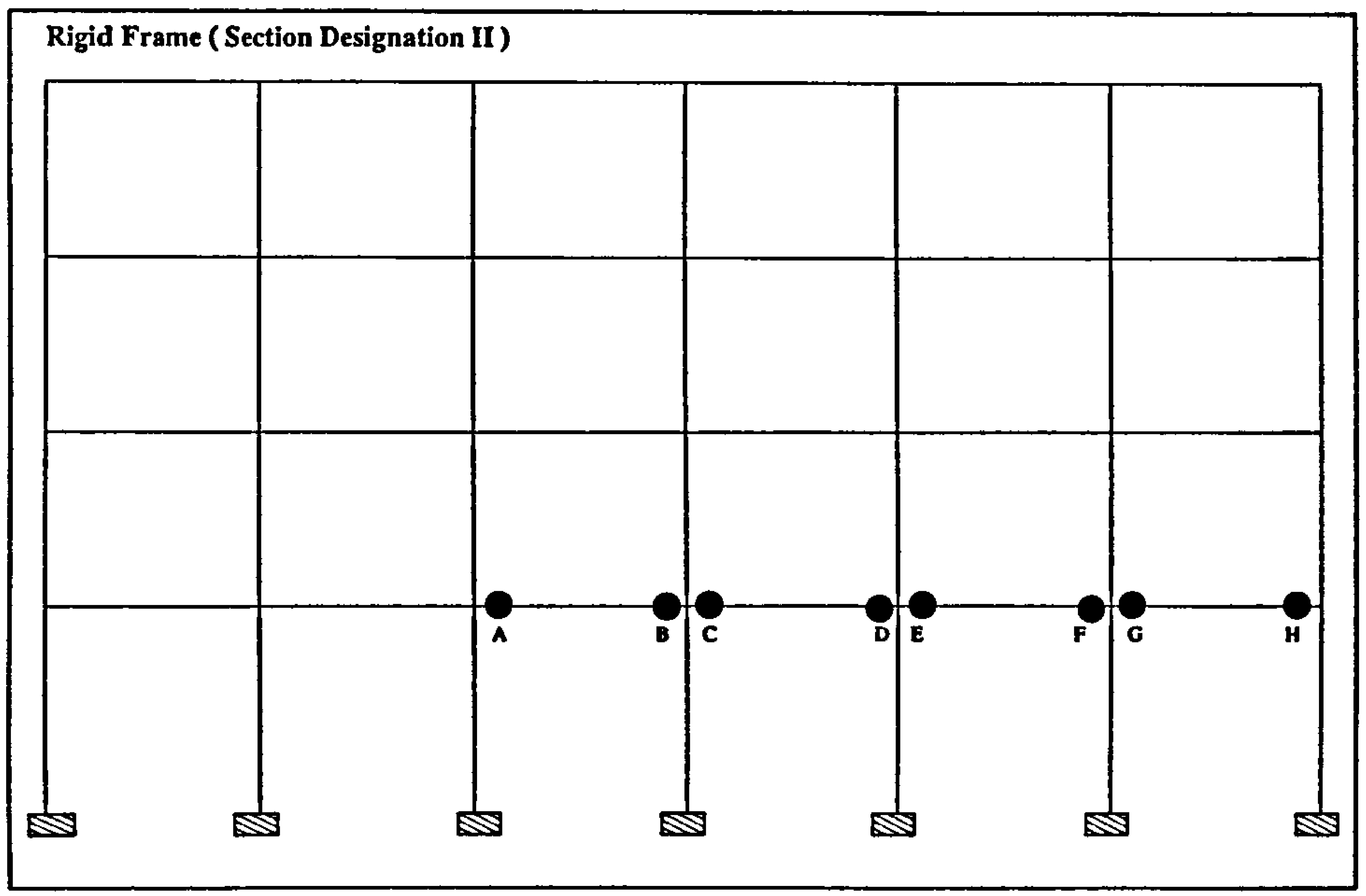


Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.34	1.17
B	1.34	1.17
C	1.34	1.17
D	1.34	1.17
E	1.34	1.17
F	1.34	1.17
G	1.34	1.17
H	N/A	1.17
I	N/A	1.17
J	N/A	1.17
K	N/A	1.16
L	N/A	1.14

Key:
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 10
Load Case 1



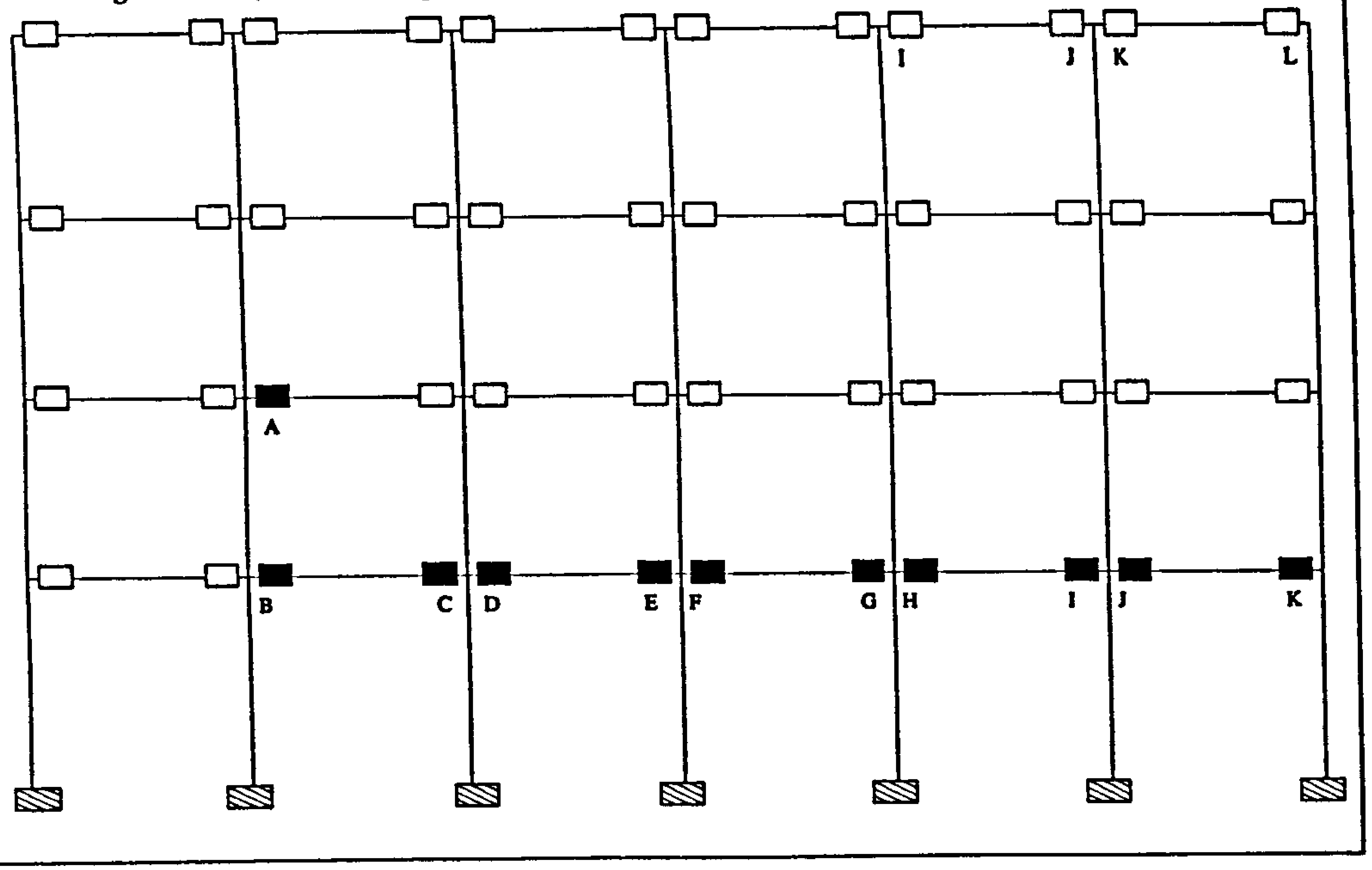


Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.65	1.43
B	1.65	1.43
C	1.65	1.43
D	1.65	1.43
E	1.65	1.43
F	1.65	1.43
G	1.65	1.43
H	1.65	1.39
I	1.65	N/A
J	1.65	N/A
K	1.65	N/A
L	1.65	N/A

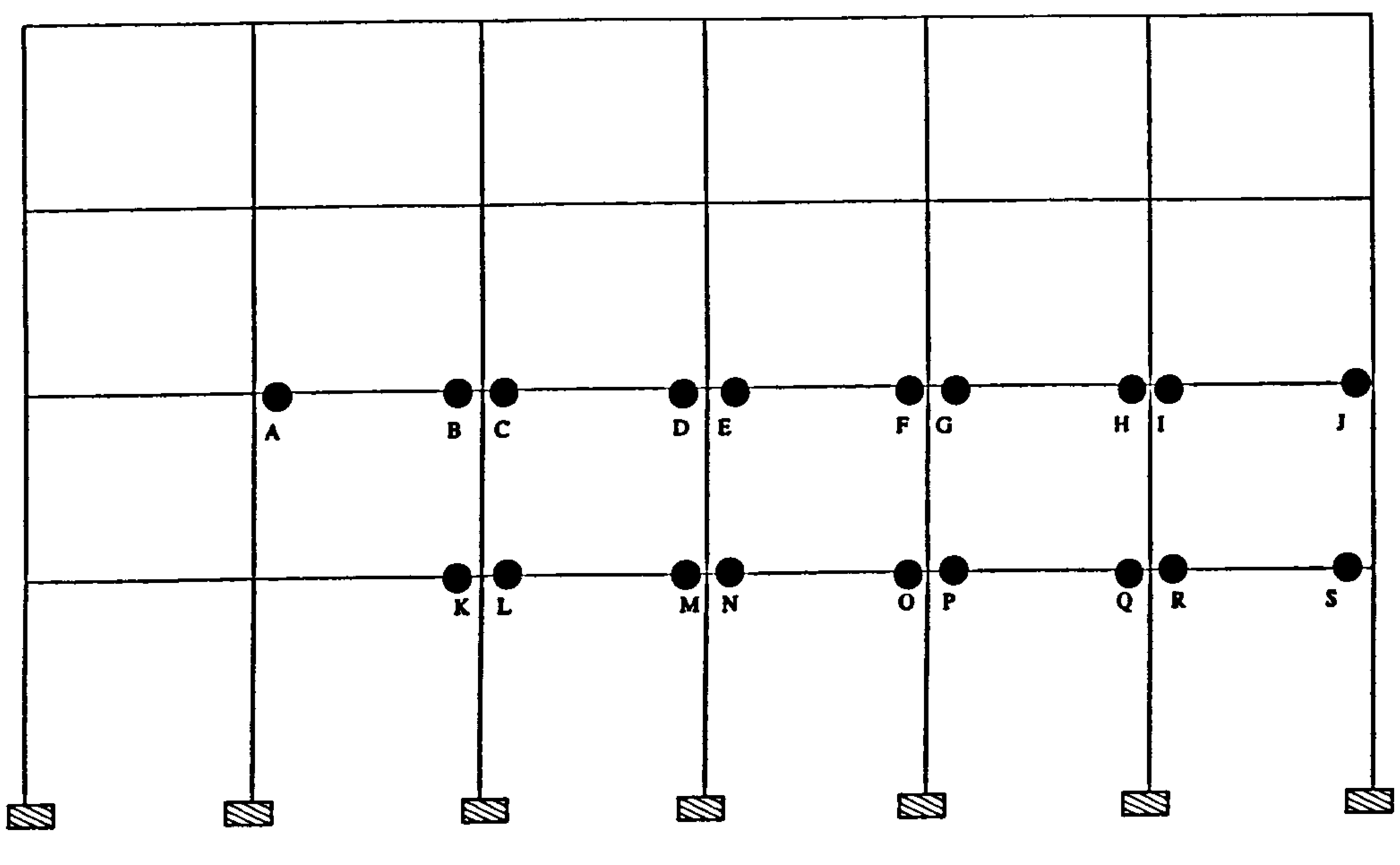
Key:
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 10
Load Case 2

Semi-Rigid Frame (Section Designation IV)



Rigid Frame (Section Designation II)



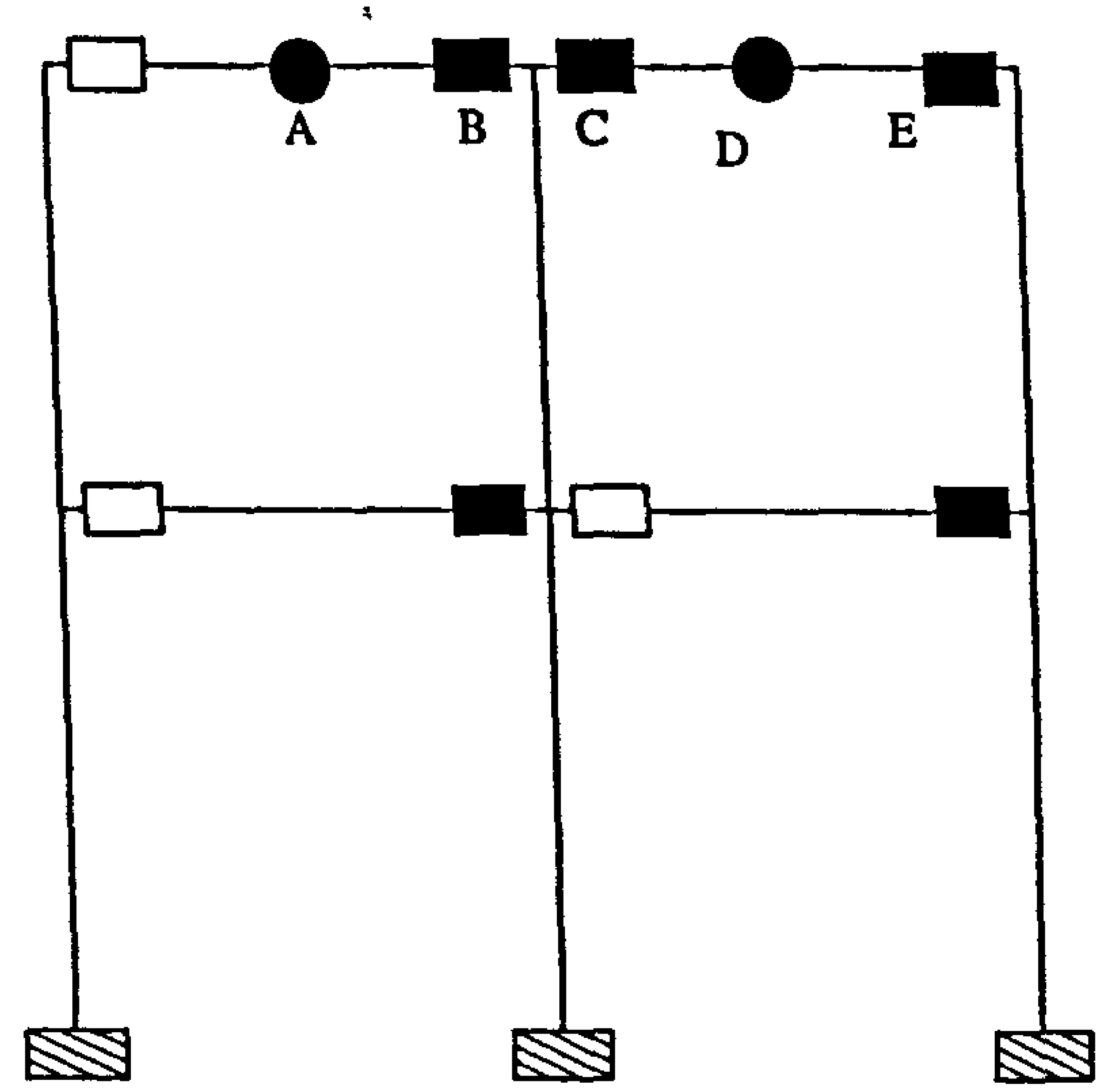
Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.98	1.85
B	1.98	1.85
C	1.98	1.85
D	1.98	1.85
E	1.98	1.85
F	1.98	1.85
G	1.98	1.85
H	1.98	1.85
I	1.98	1.84
J	1.98	1.84
K	1.98	1.84
L	N/A	1.82
M	N/A	1.82
N	N/A	1.83
O	N/A	1.83
P	N/A	1.83
Q	N/A	1.83
R	N/A	1.81
S	N/A	1.76

Key:

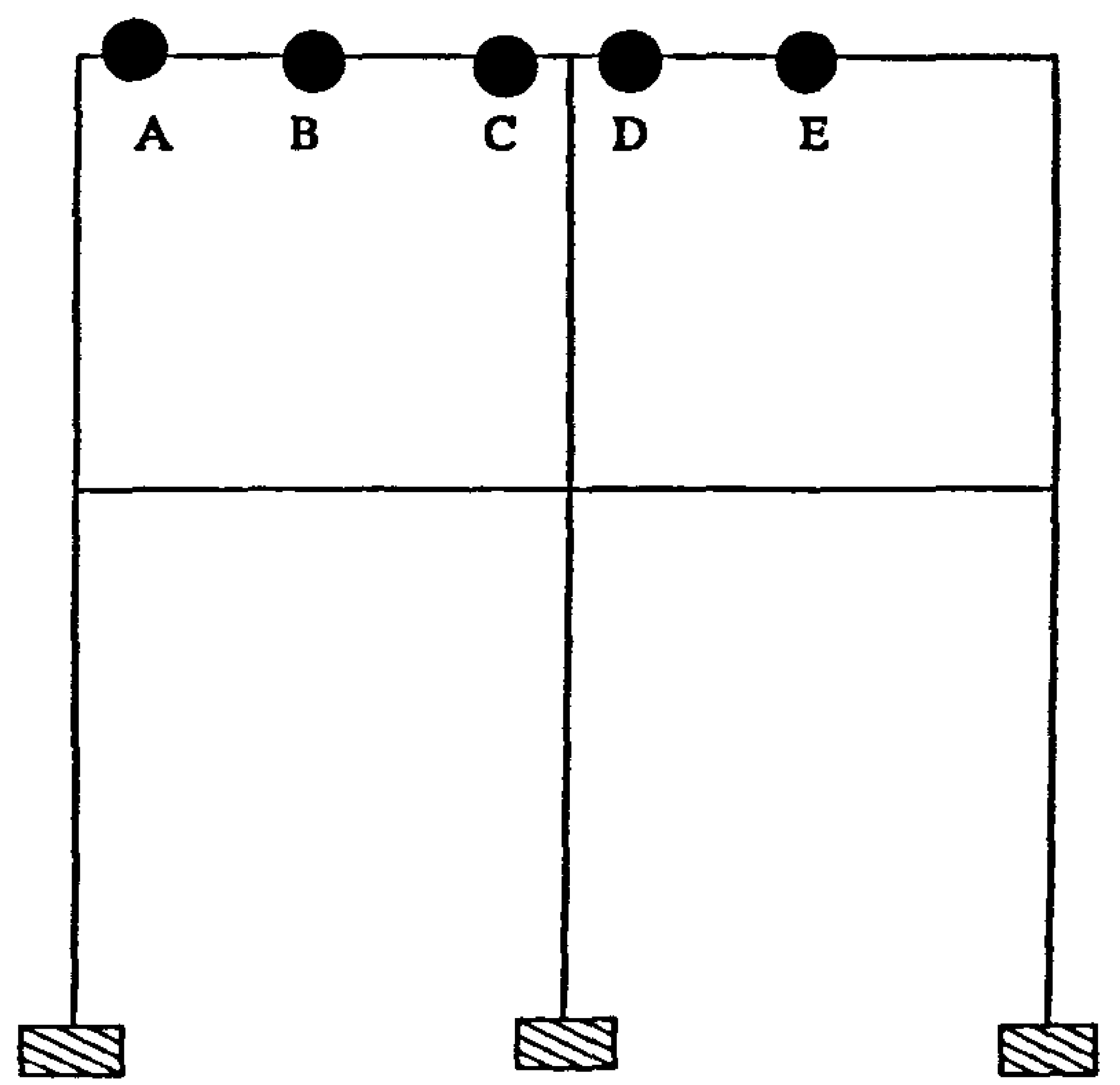
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 10
Load Case 3

Semi-Rigid Frame (Section Designation III)



Rigid Frame (Section Designation III)



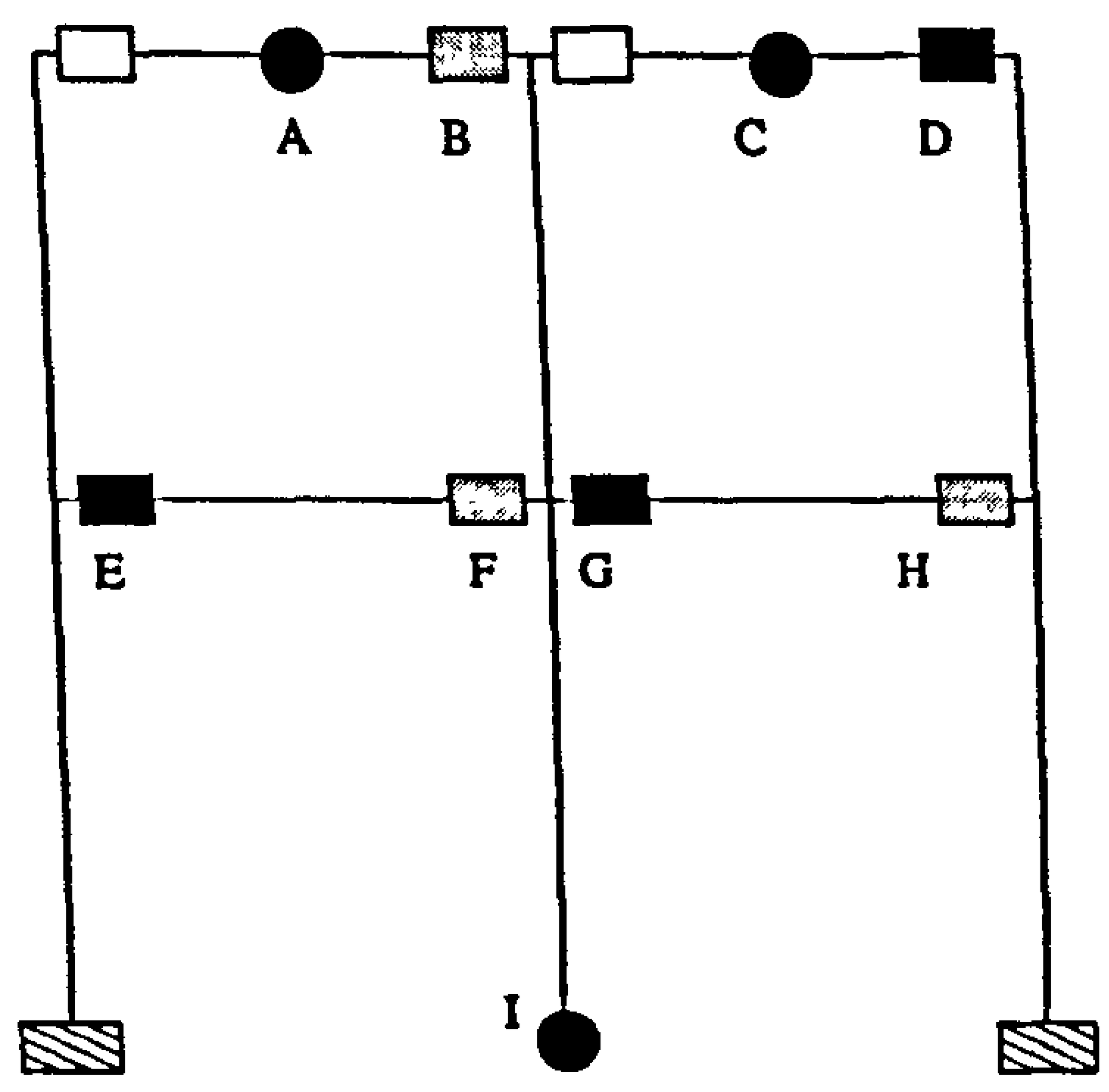
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.40	2.03
B	1.40	1.78
C	1.40	1.66
D	1.44	1.83
E	1.40	1.80
F	1.45	N/A

Key:

- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

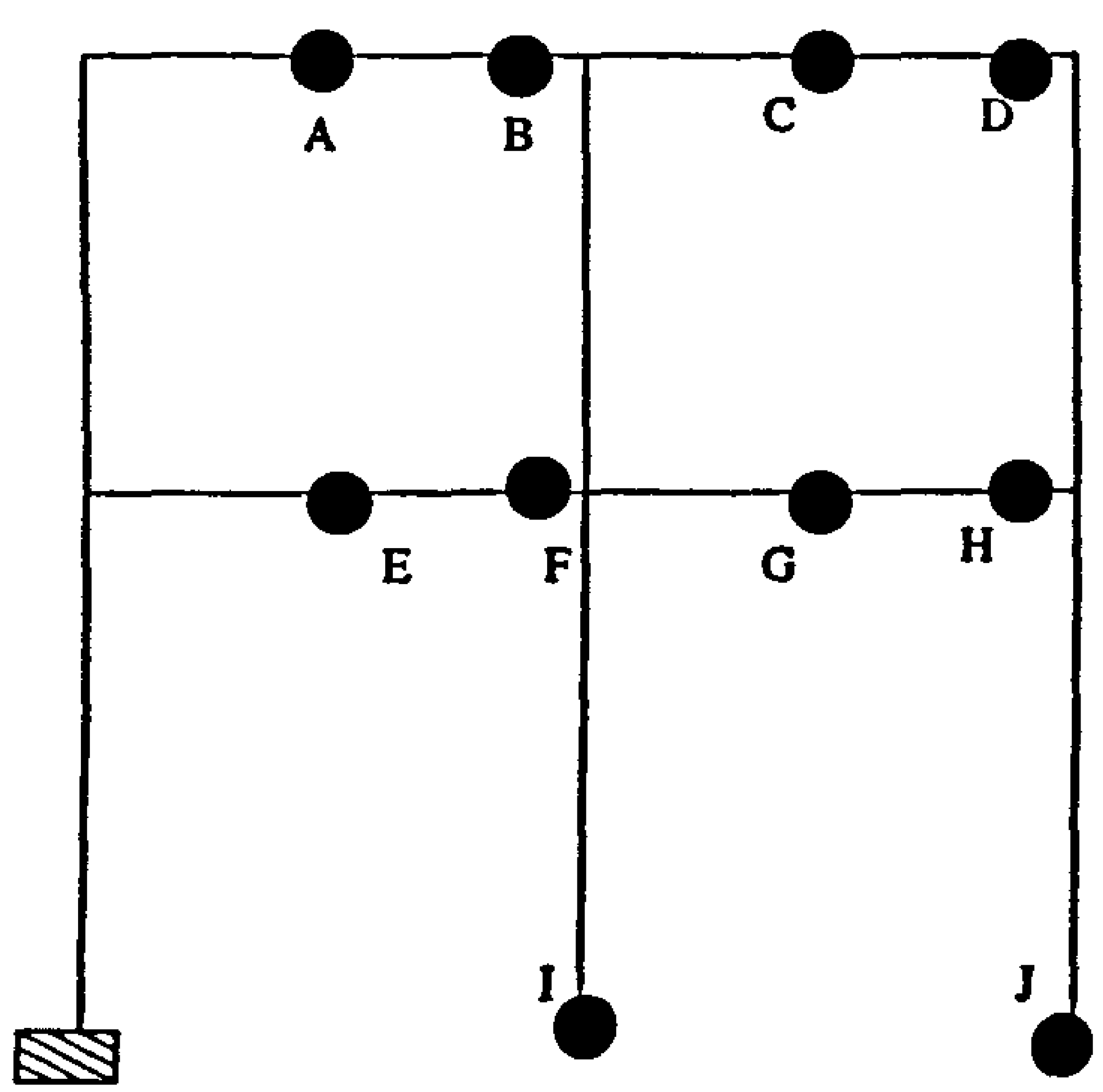
Frame 11
Load Case 1

Semi-Rigid Frame (Section Designation III)



Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.45	1.97
B	< 1.00	1.60
C	1.48	2.11
D	1.45	2.14
E	1.48	2.27
F	< 1.00	1.71
G	1.48	2.28
H	< 1.00	2.15
I	N/A	2.23
J	N/A	2.28

Rigid Frame(Section Designation III)

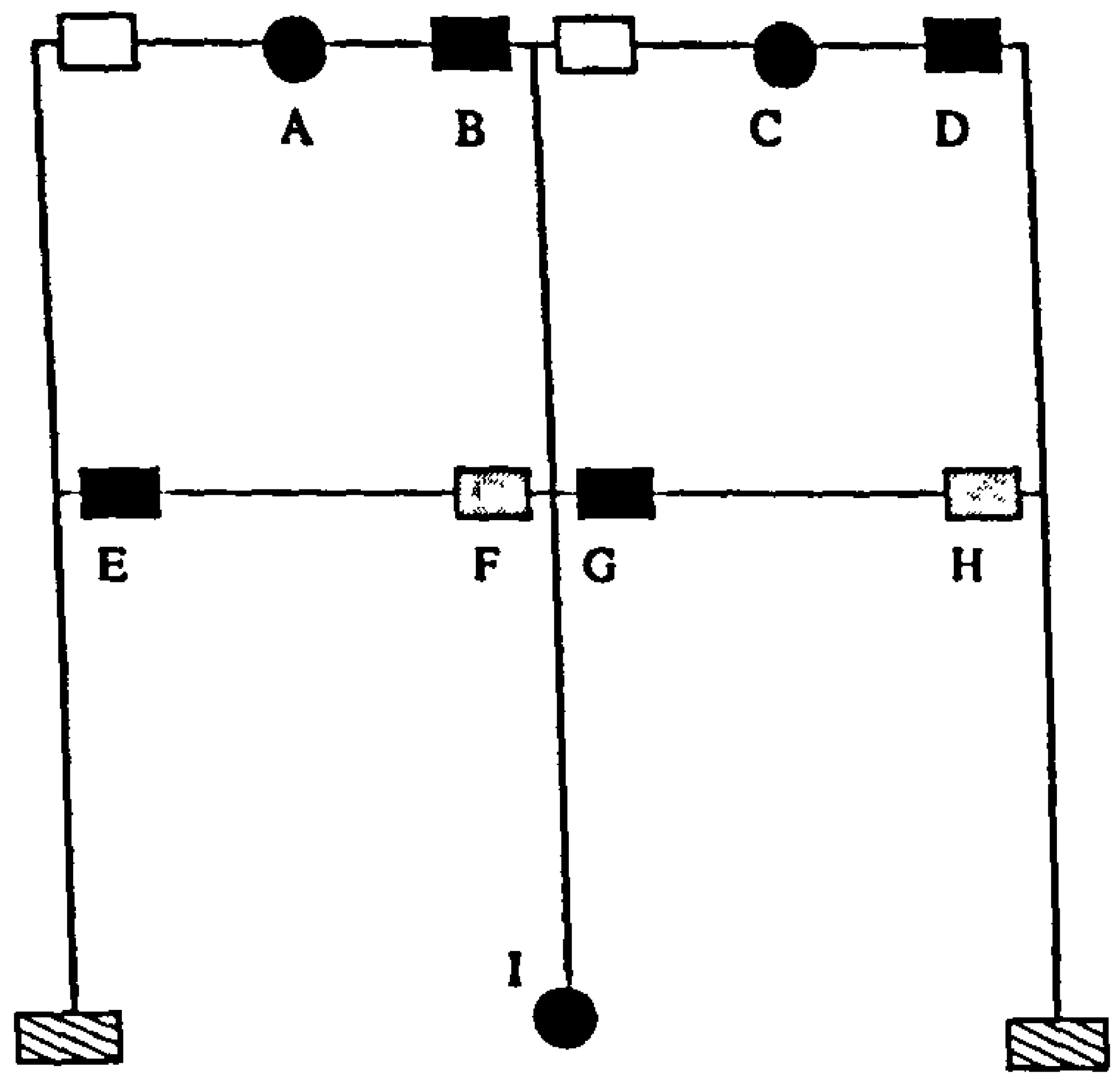


Key:

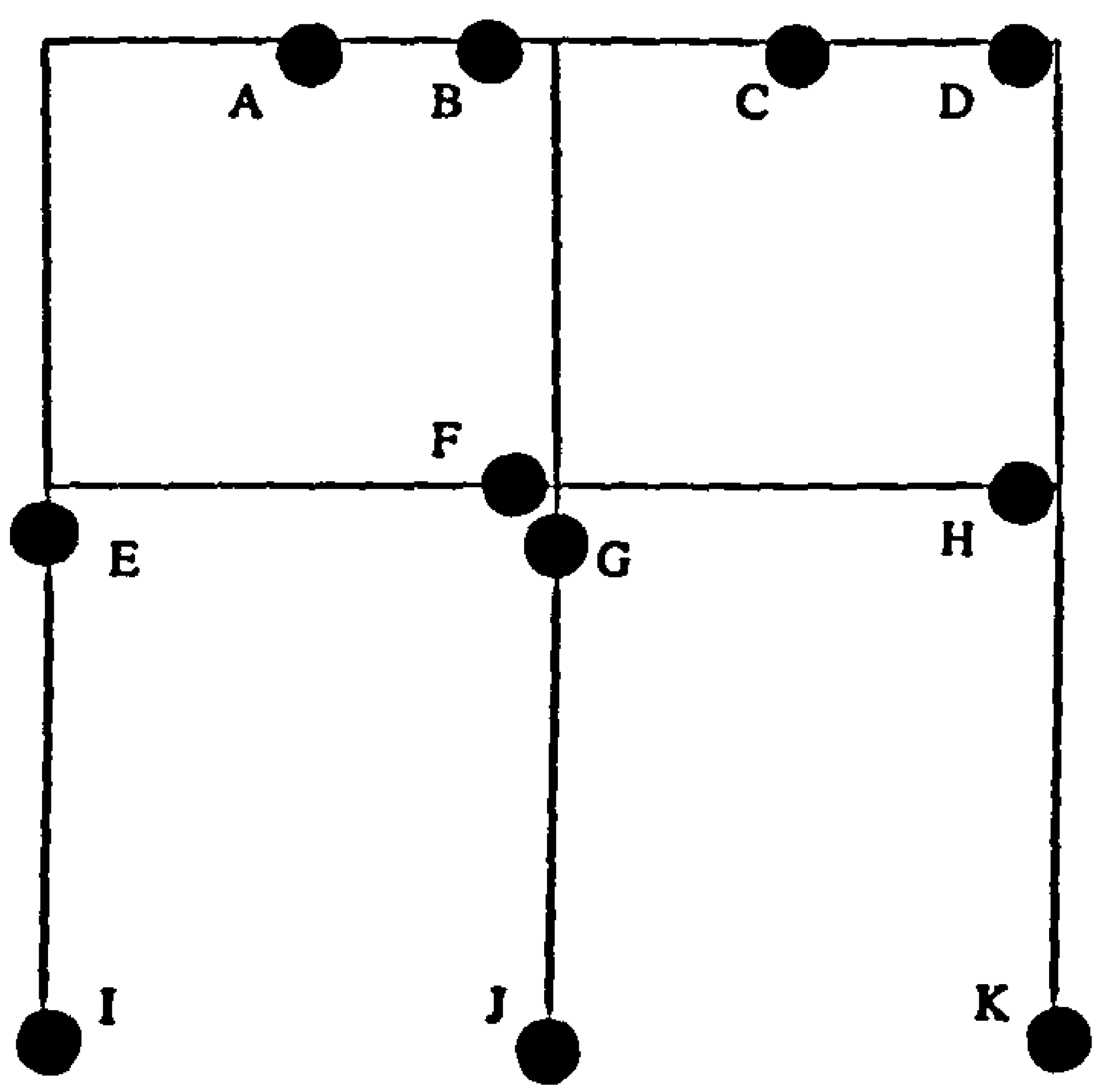
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 11
Load Case 2

Semi-Rigid Frame (Section Designation III)



Rigid Frame (Section Designation III)

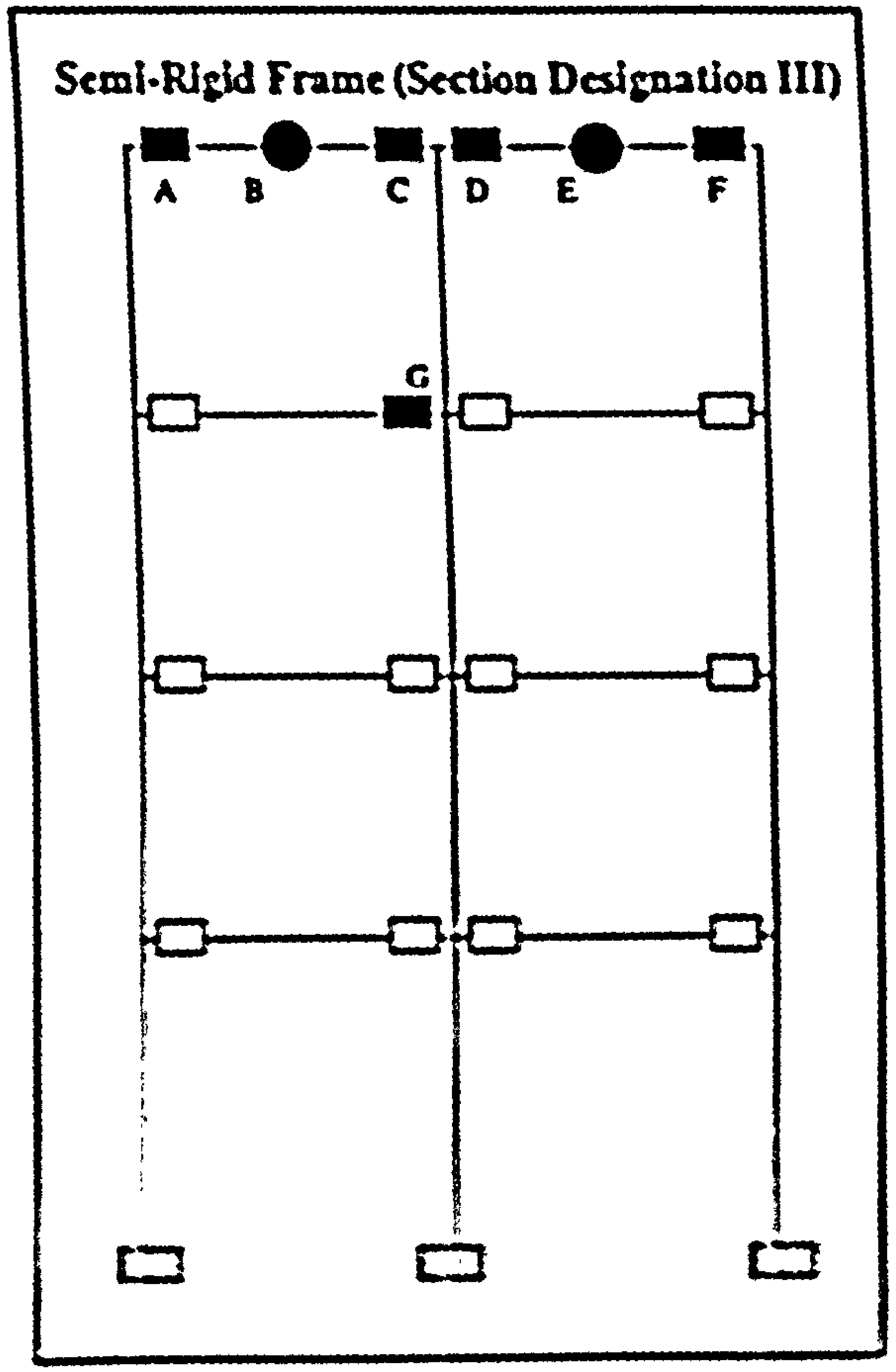


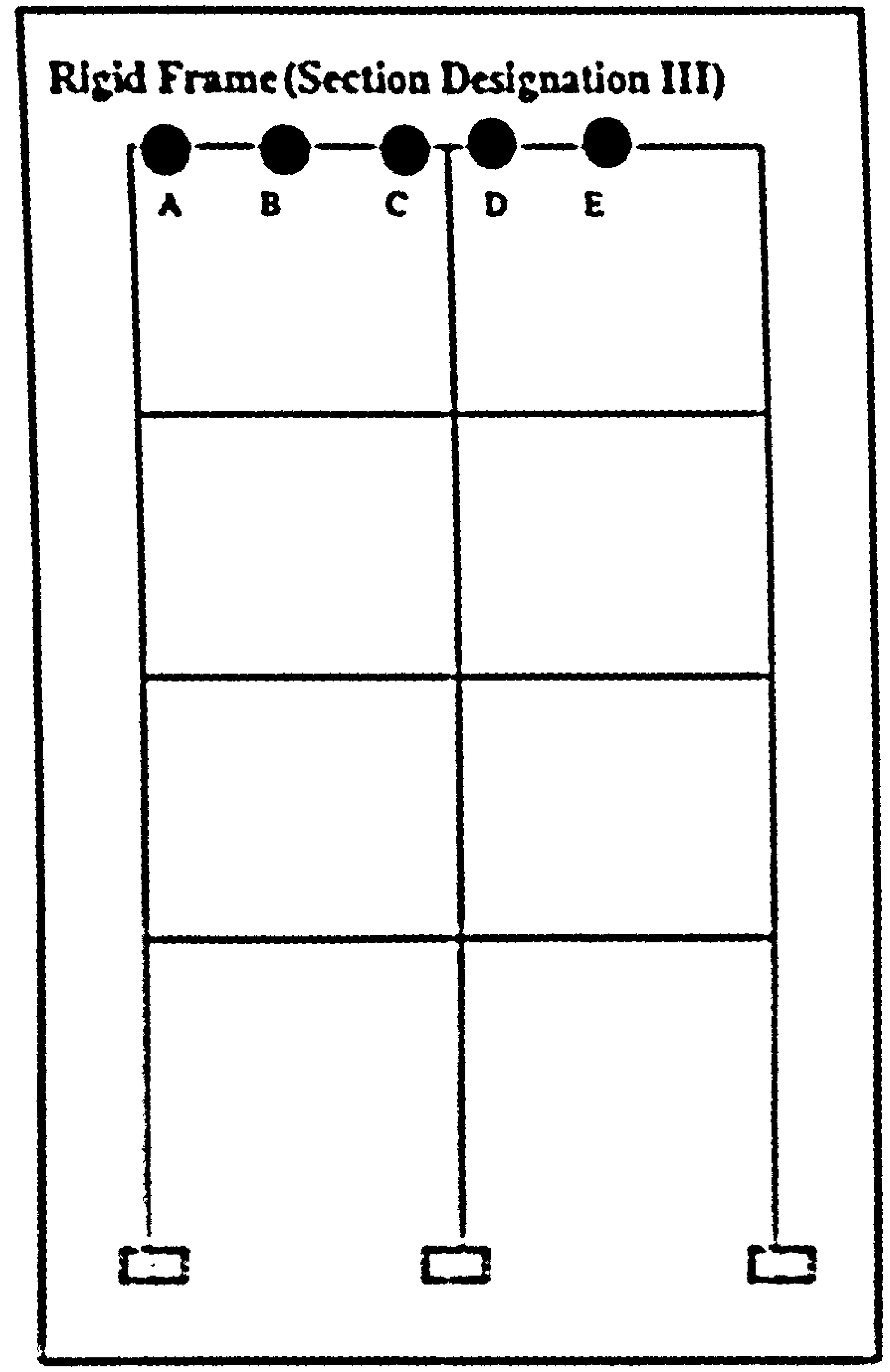
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.59	2.27
B	1.57	1.75
C	1.60	2.41
D	1.57	2.40
E	1.57	2.41
F	< 1.00	2.11
G	1.57	2.41
H	< 1.00	2.38
I	N/A	2.39
J	N/A	2.16
K	N/A	2.33

Key:

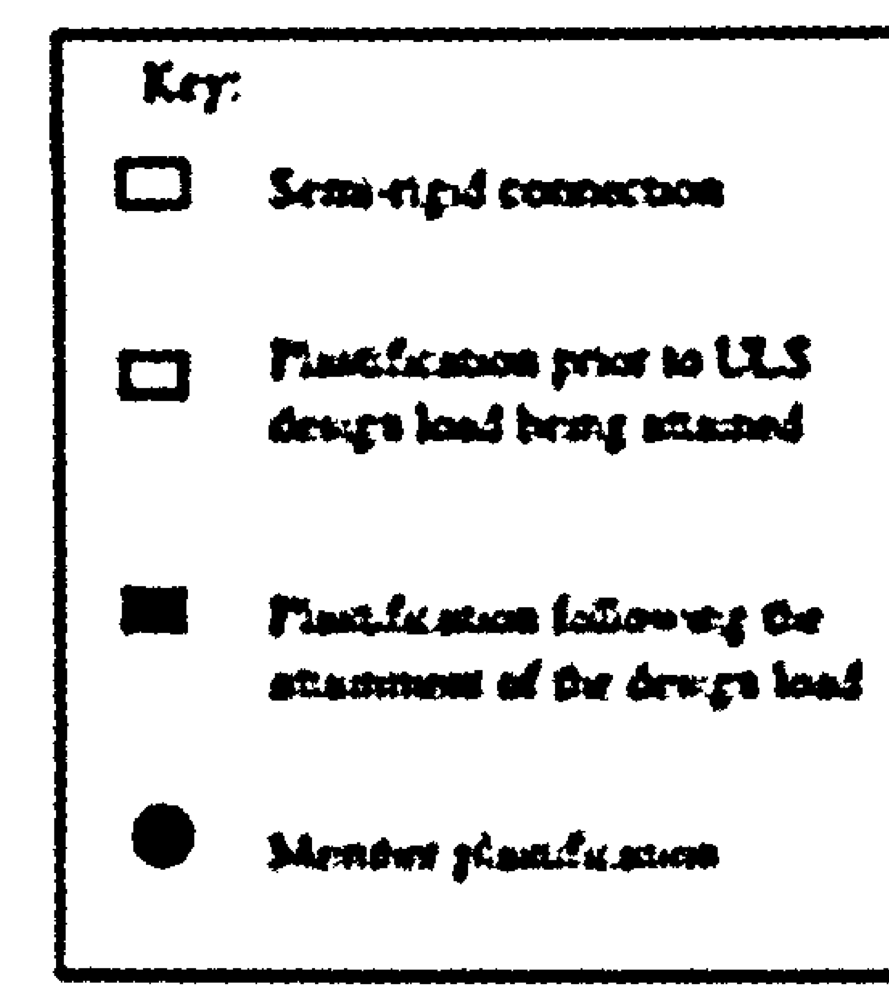
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 11
Load Case 3





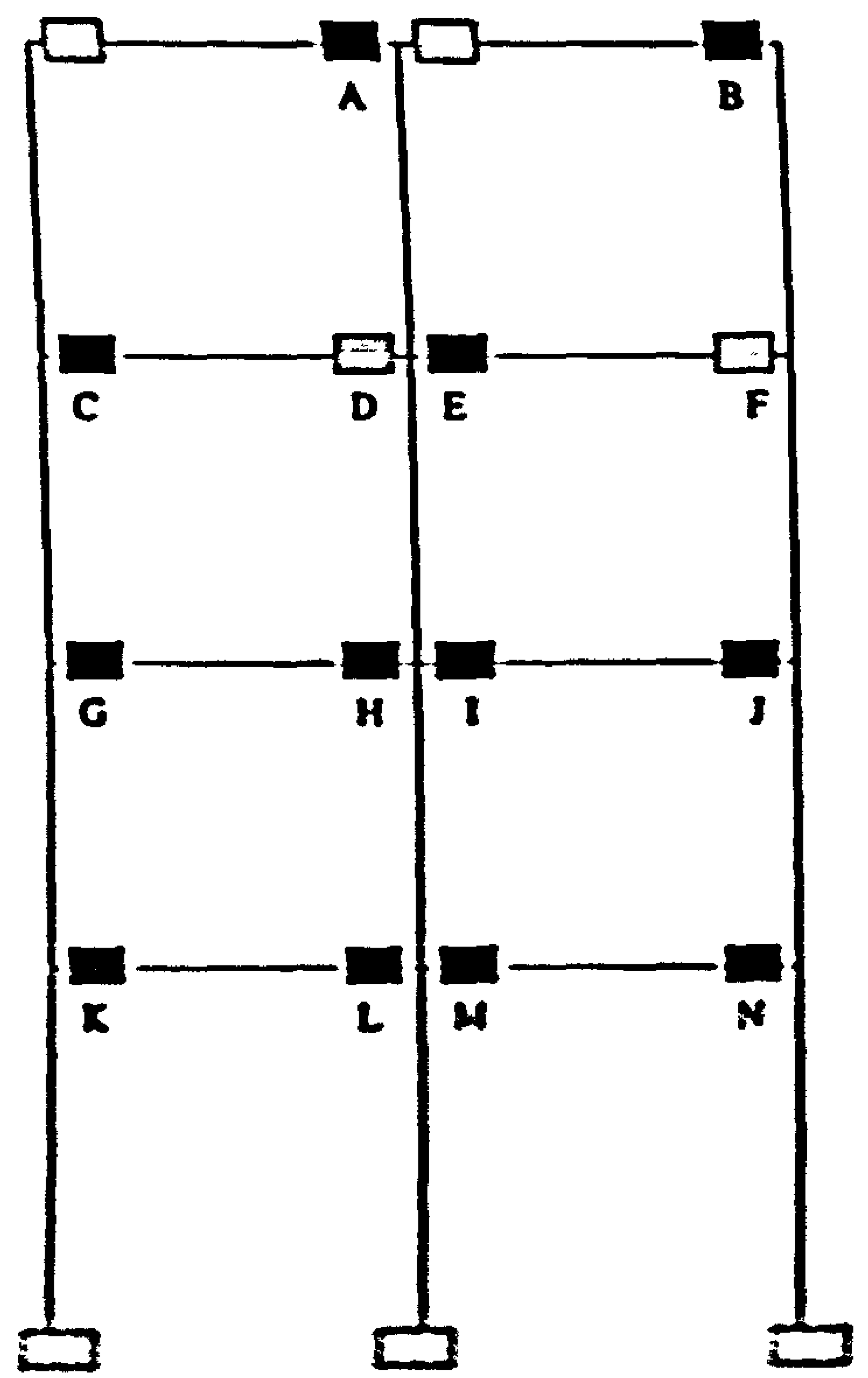
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.67	2.34
B	1.65	2.09
C	1.65	1.98
D	1.65	2.13
E	1.67	2.12
F	1.67	N/A
G	1.65	N/A



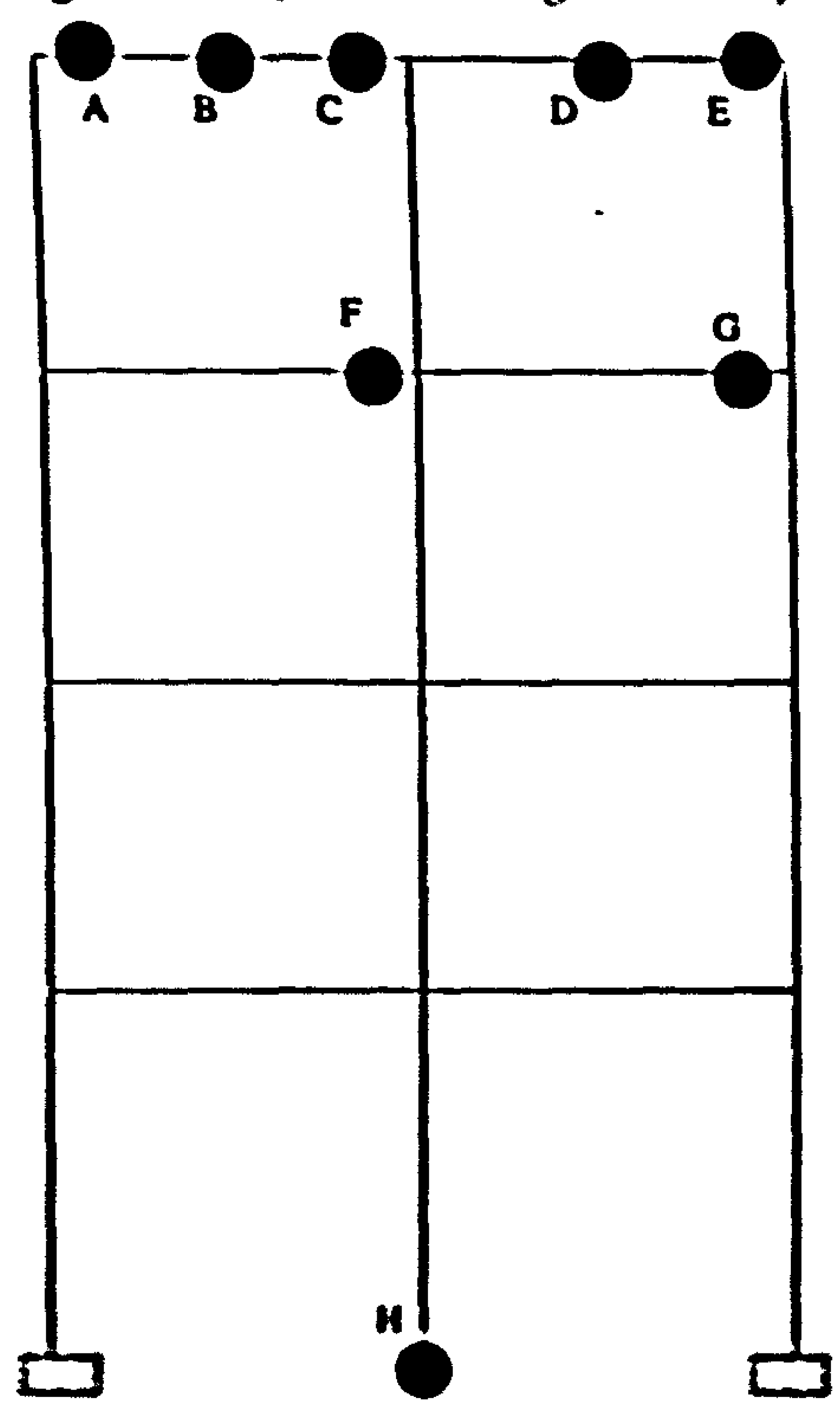
Frame 12
Load Case 1

Plastic-hinges in minor-axis "wind-moment" frames.

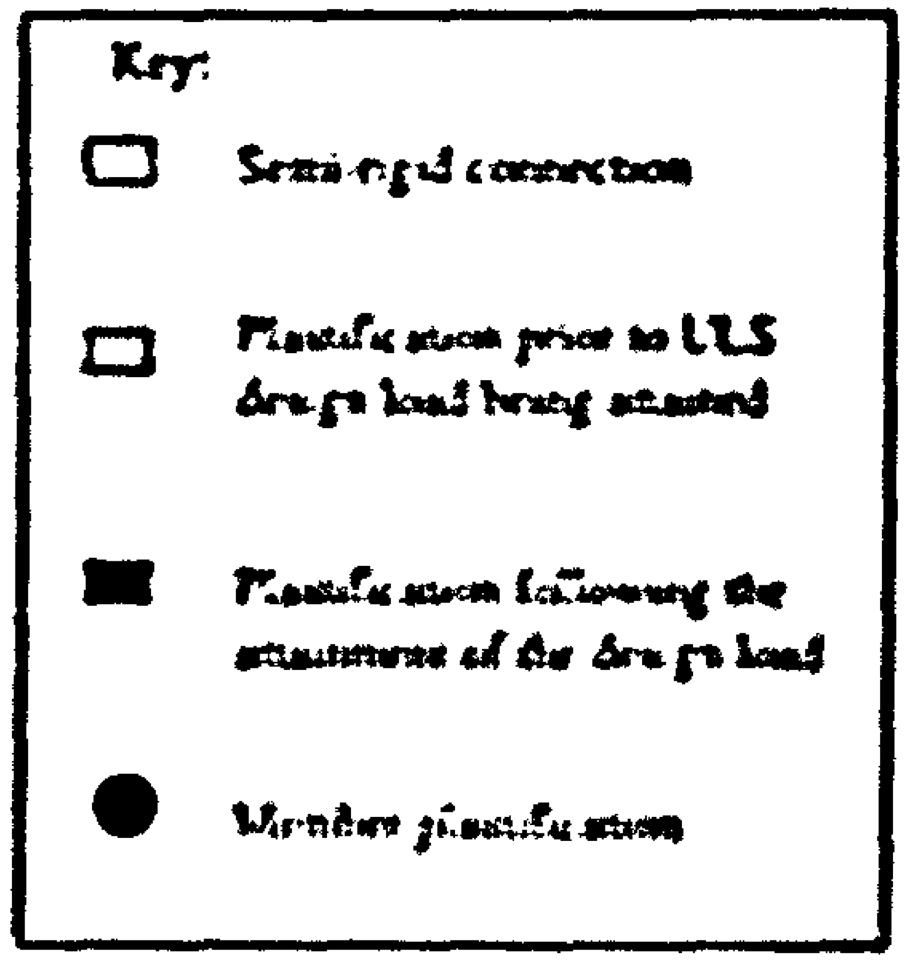
Semi-Rigid Frame (Section Designation III)



Rigid Frame(Section Designation III)

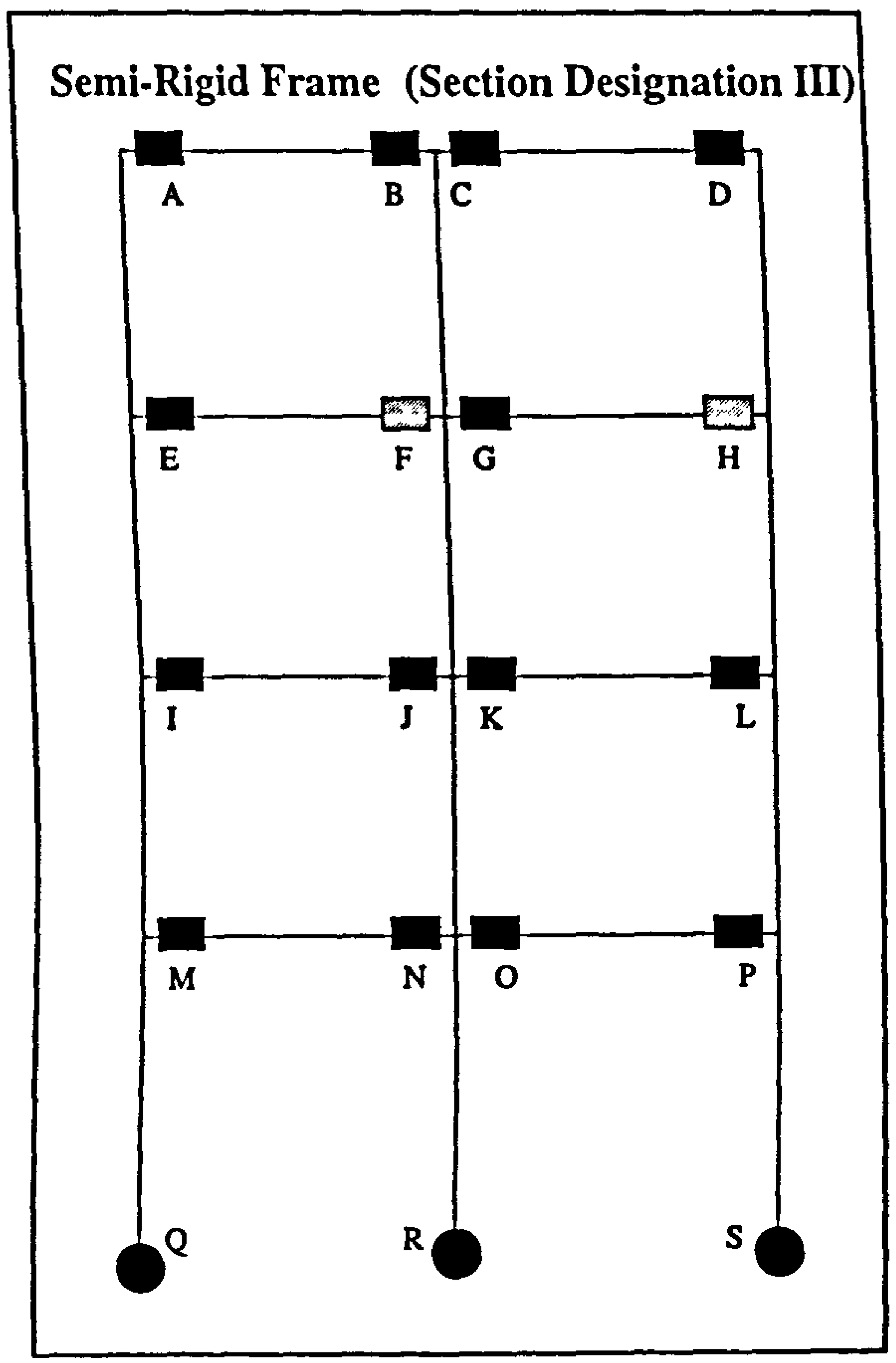


Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.49	2.73
B	1.49	2.29
C	1.49	1.81
D	< 1.00	2.44
E	1.49	2.40
F	< 1.00	2.32
G	1.49	2.59
H	1.49	2.72
I	1.49	N/A
J	1.49	N/A
K	1.49	N/A
L	1.49	N/A
M	1.49	N/A
N	1.49	N/A

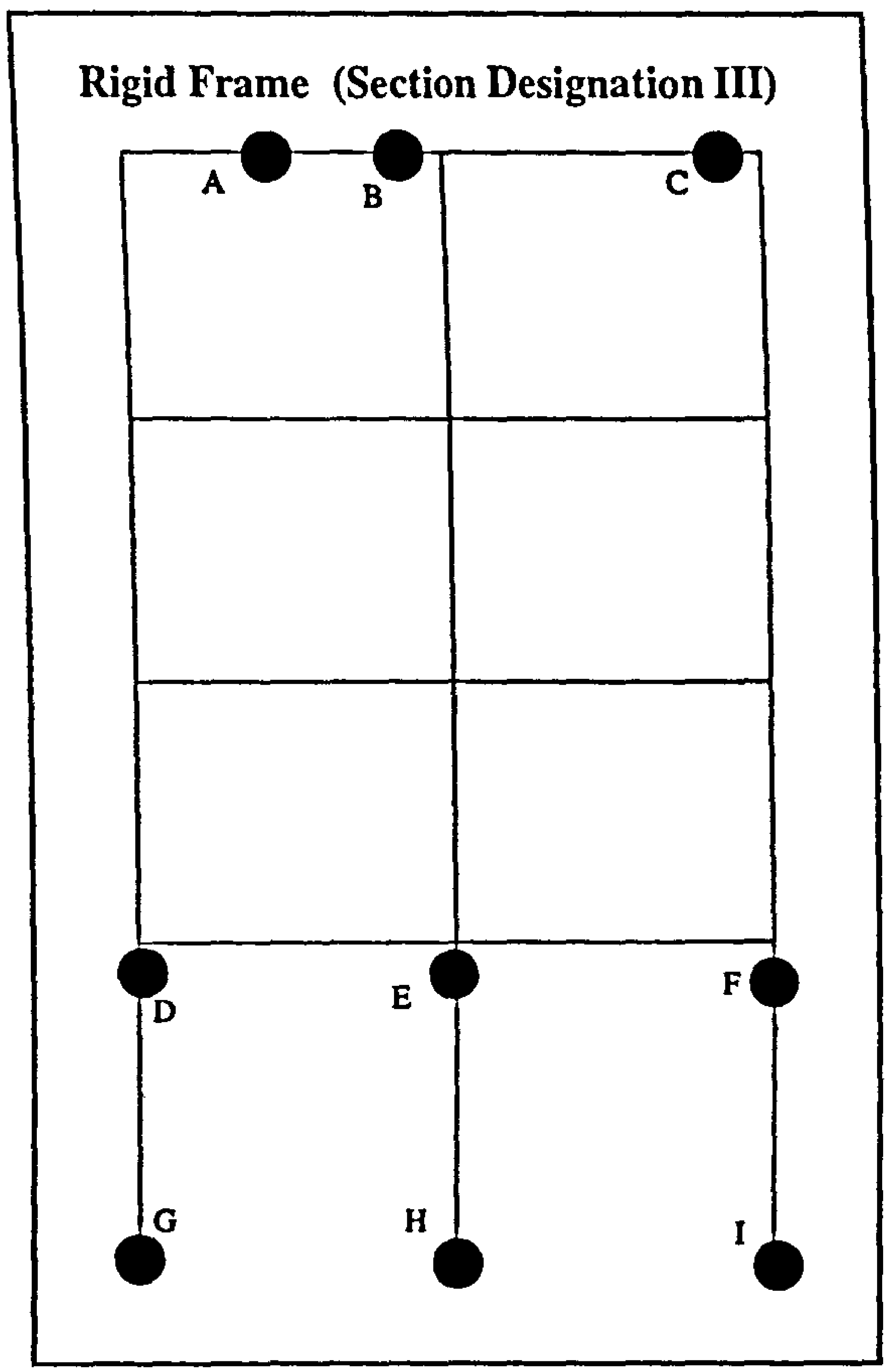


**Frame 12
Load Case 2**

Semi-Rigid Frame (Section Designation III)



Rigid Frame (Section Designation III)



Frame 12
Load Case 3

Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.45	2.62
B	1.45	1.96
C	1.45	2.64
D	< 1.00	2.69
E	1.45	2.66
F	< 1.00	2.69
G	1.45	2.62
H	1.45	2.43
I	1.45	2.58
J	1.45	N/A
K	1.45	N/A
L	1.45	N/A
M	1.45	N/A
N	1.45	N/A
O	1.45	N/A
P	1.45	N/A
Q	1.45	N/A
R	1.45	N/A
S	1.45	N/A

Key:

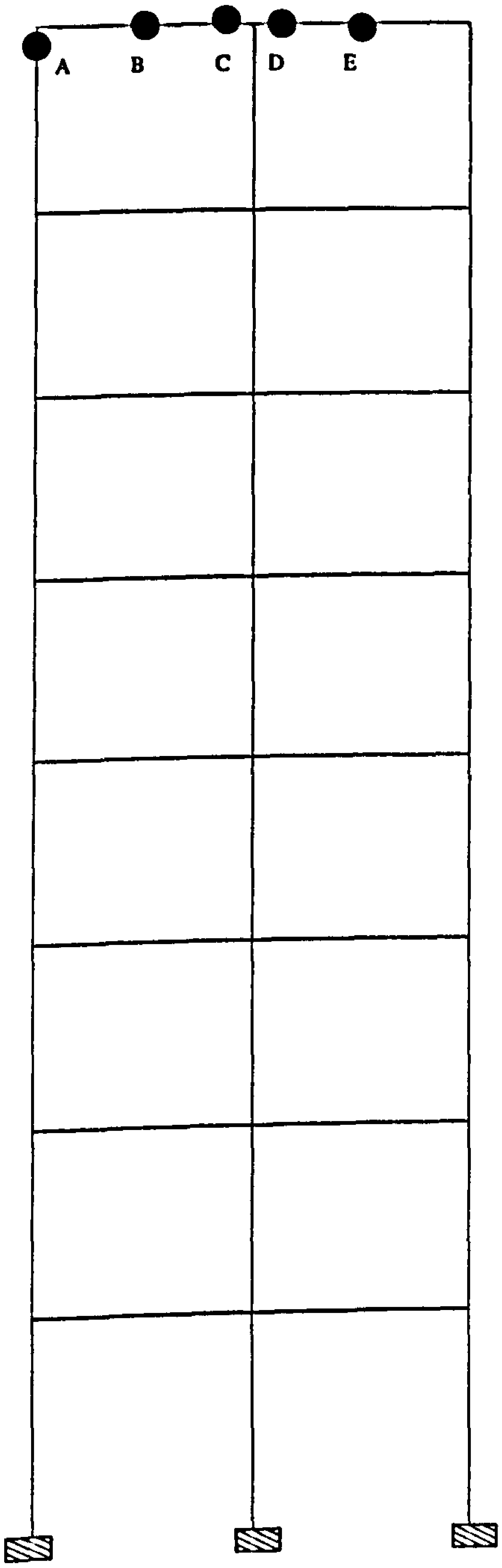
□ Semi-rigid connection

▨ Plastification prior to ULS design load being attained

■ Plastification following the attainment of the design load

● Member plastification

Rigid Frame
(Section Designation I)



Hinge Loaction	Load Level at Hinge Formation Rigid
A	1.89
B	1.71
C	1.60
D	1.76
E	1.73

Key:

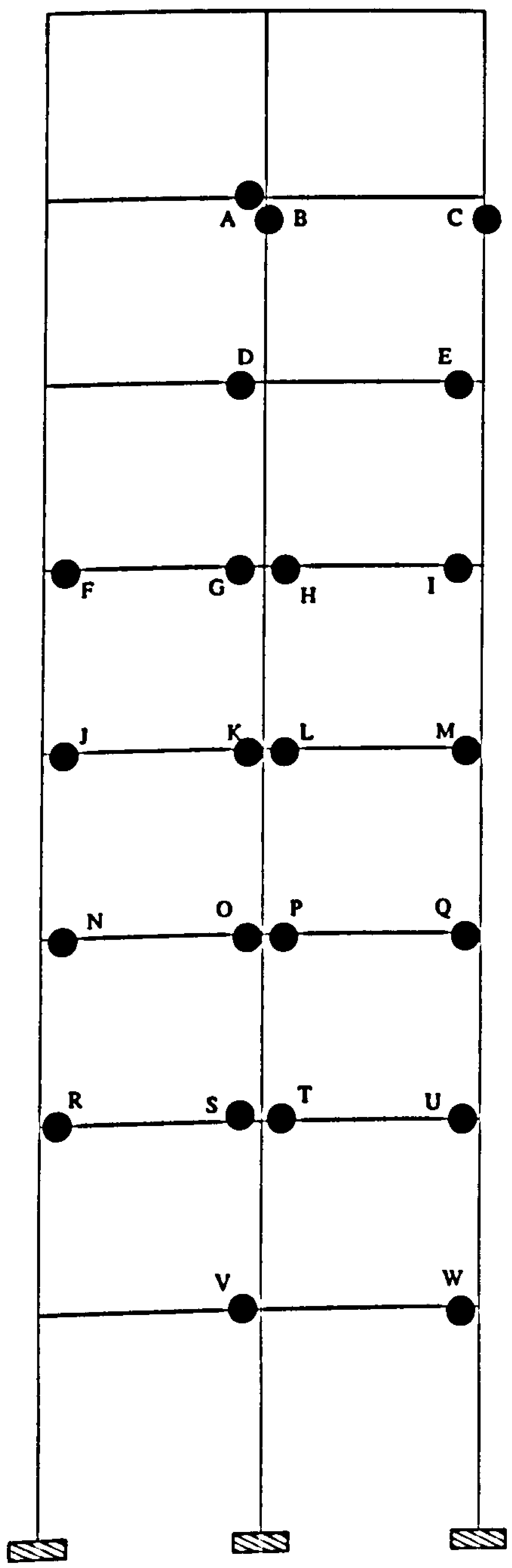
Plastification prior to ULS design load being attained

Plastification following the attainment of the design load

Member plastification

Frame 13
Load Case 1

Rigid Frame
(Section Designation I)



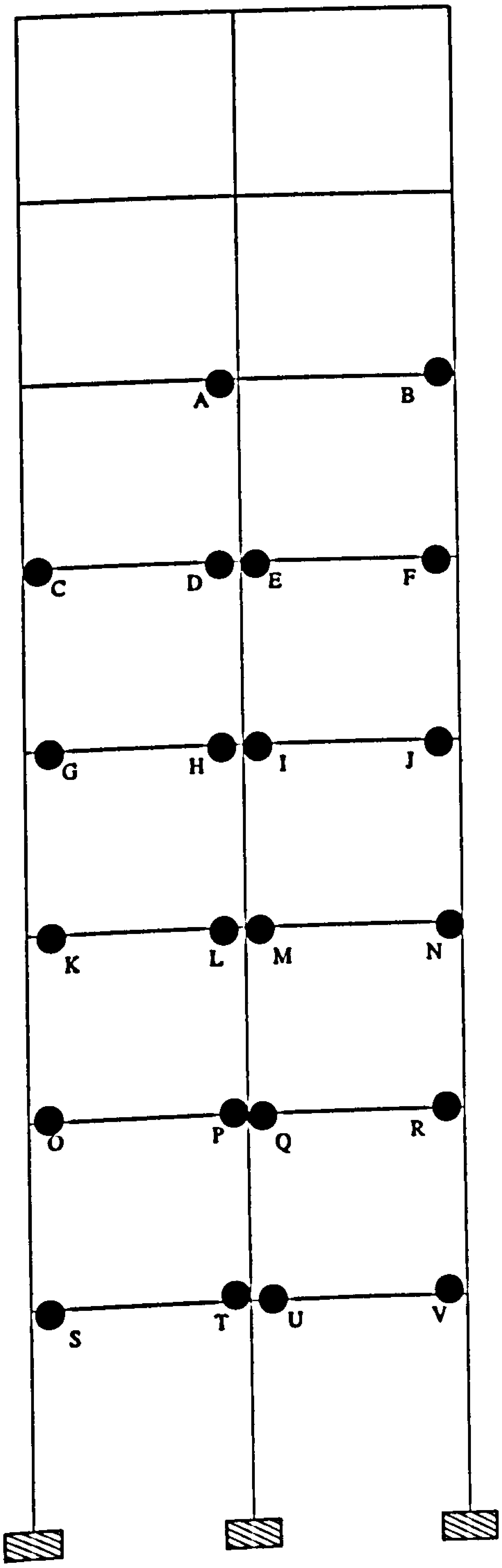
Hinge Loaction	Load Level at Hinge Formation Rigid
A	1.23
B	1.34
C	1.33
D	1.04
E	1.15
F	1.33
G	1.04
H	1.33
I	1.07
J	1.31
K	1.14
L	1.31
M	1.18
N	1.33
O	1.12
P	1.33
Q	1.18
R	1.34
S	1.16
T	1.34
U	1.21
V	1.17
W	1.25

Key:

- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 13
Load Case 2

Rigid Frame
(Section Designation I)

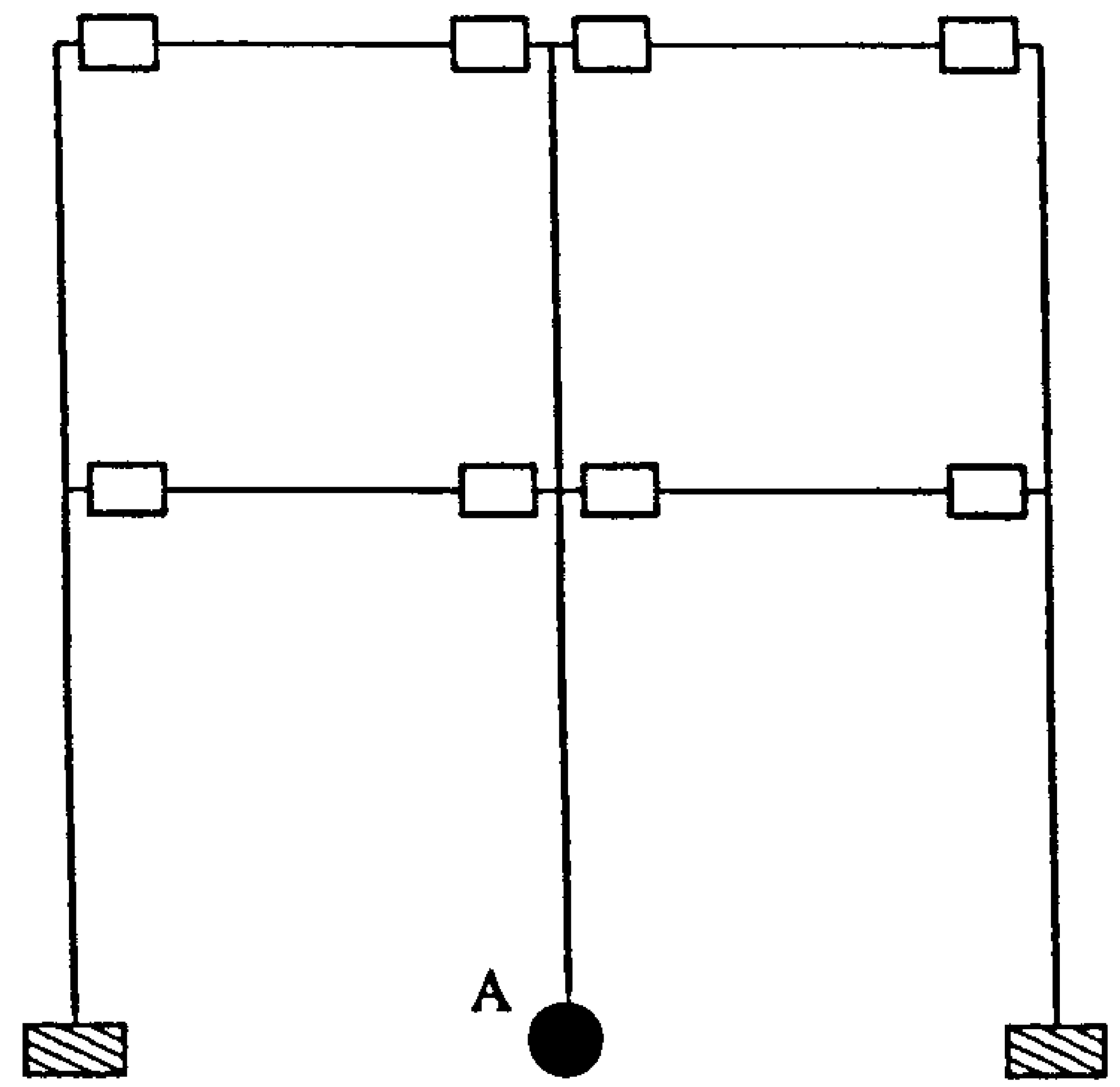


Hinge Loaction	Load Level at Hinge Formation Rigid
A	1.04
B	1.12
C	1.20
D	0.98
E	1.20
F	1.01
G	1.18
H	1.05
I	1.19
J	1.08
K	1.19
L	1.05
M	1.19
N	1.09
O	1.20
P	1.08
R	1.19
S	1.12
T	1.20
U	1.08
V	1.20
W	1.14

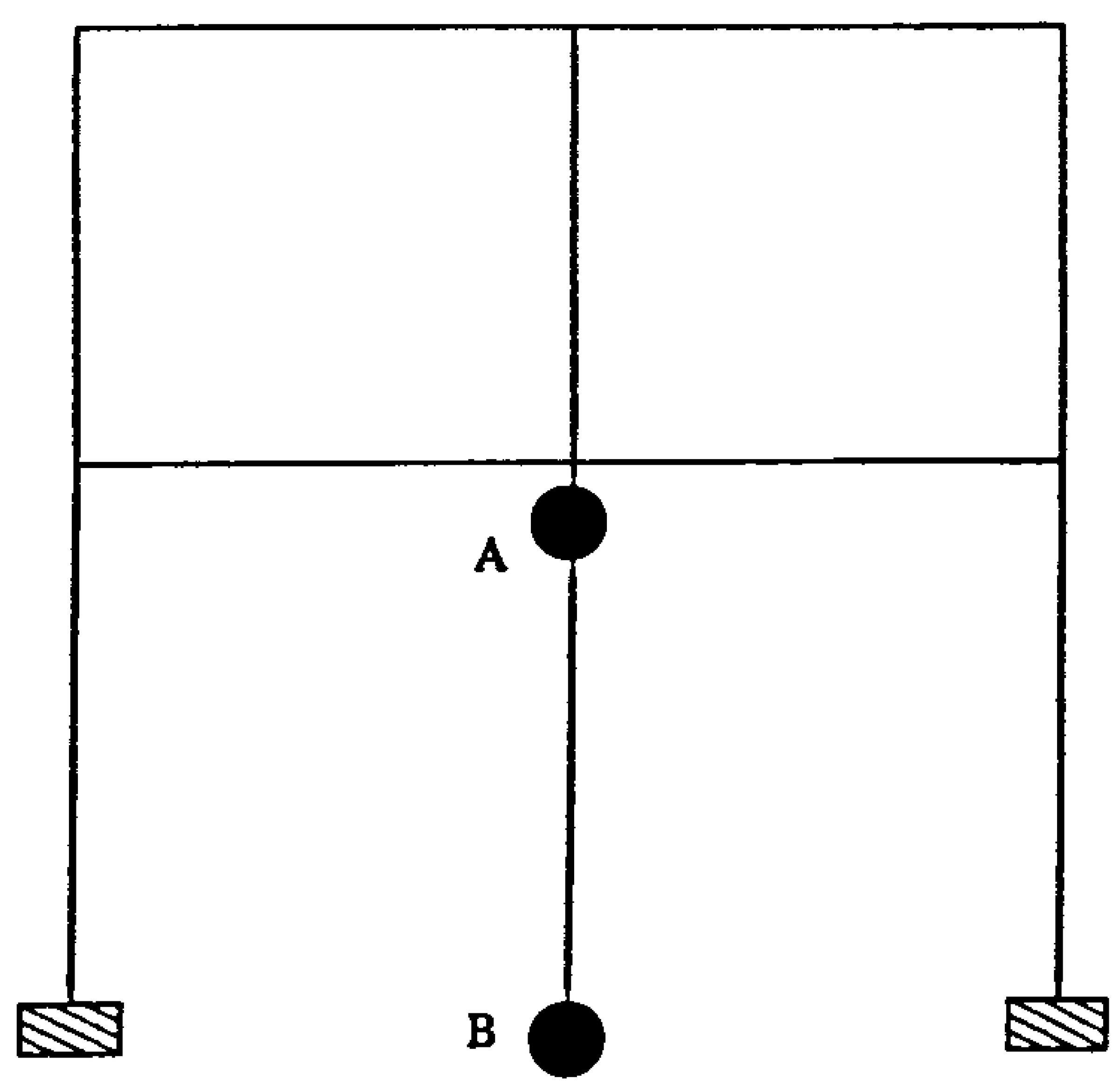
Key:
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 13
Load Case 3

Semi-Rigid Frame (Section Designation II)



Rigid Frame (Section Designation II)



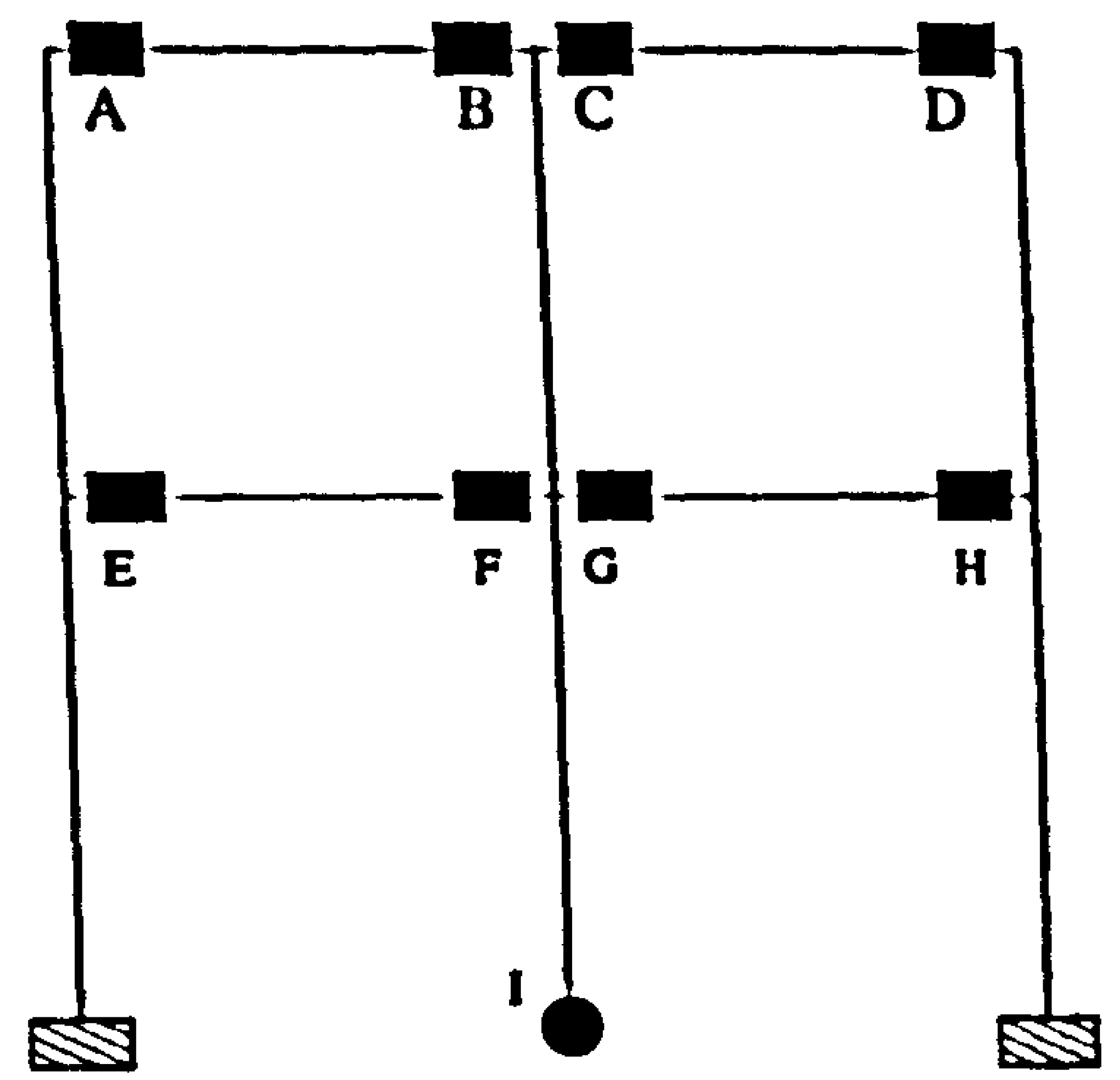
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	4.84	5.65
B	N/A	5.60

Key:

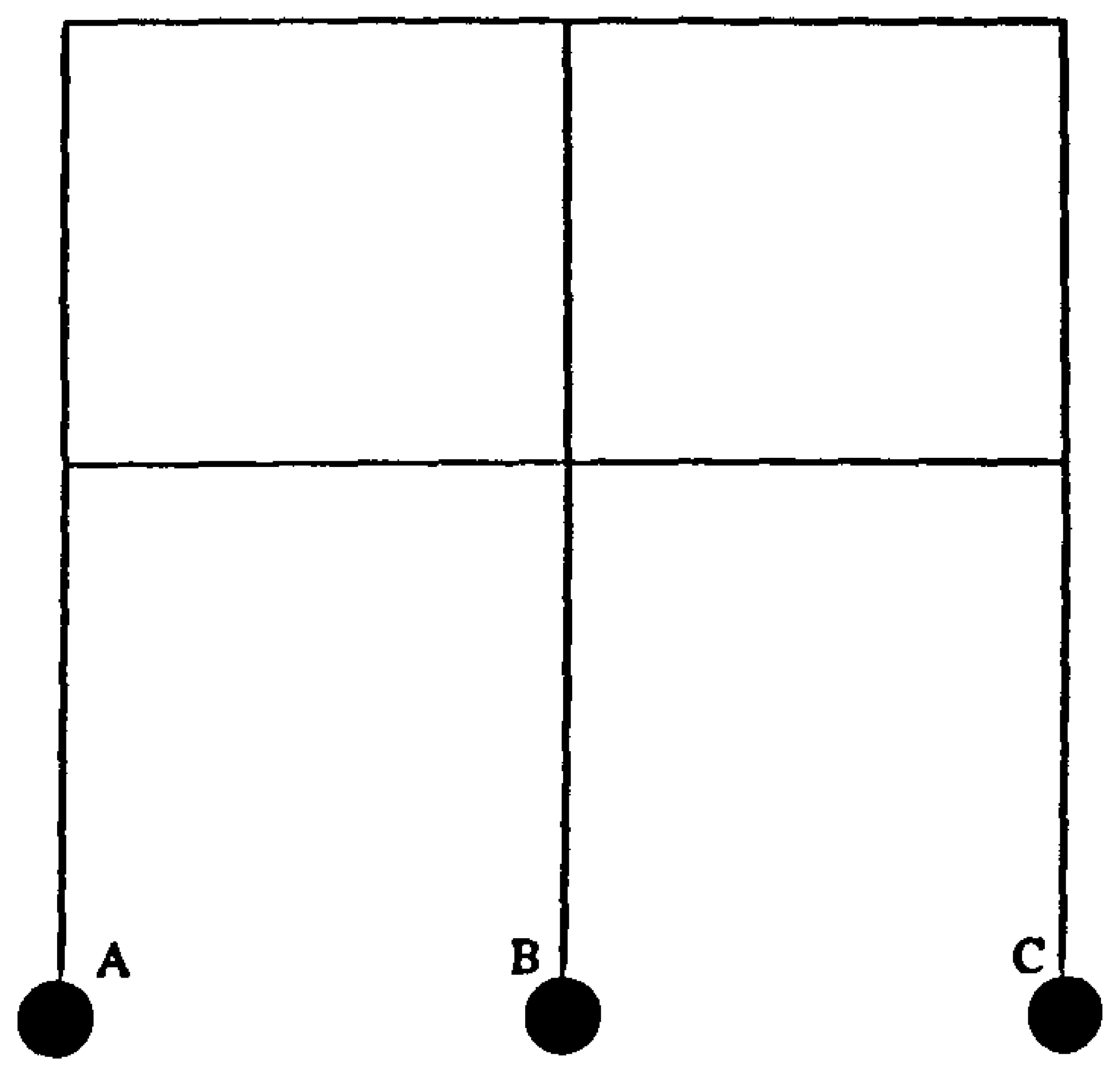
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 14
Load Case 1

Semi-Rigid Frame (Section Designation II)



Rigid Frame (Section Designation II)



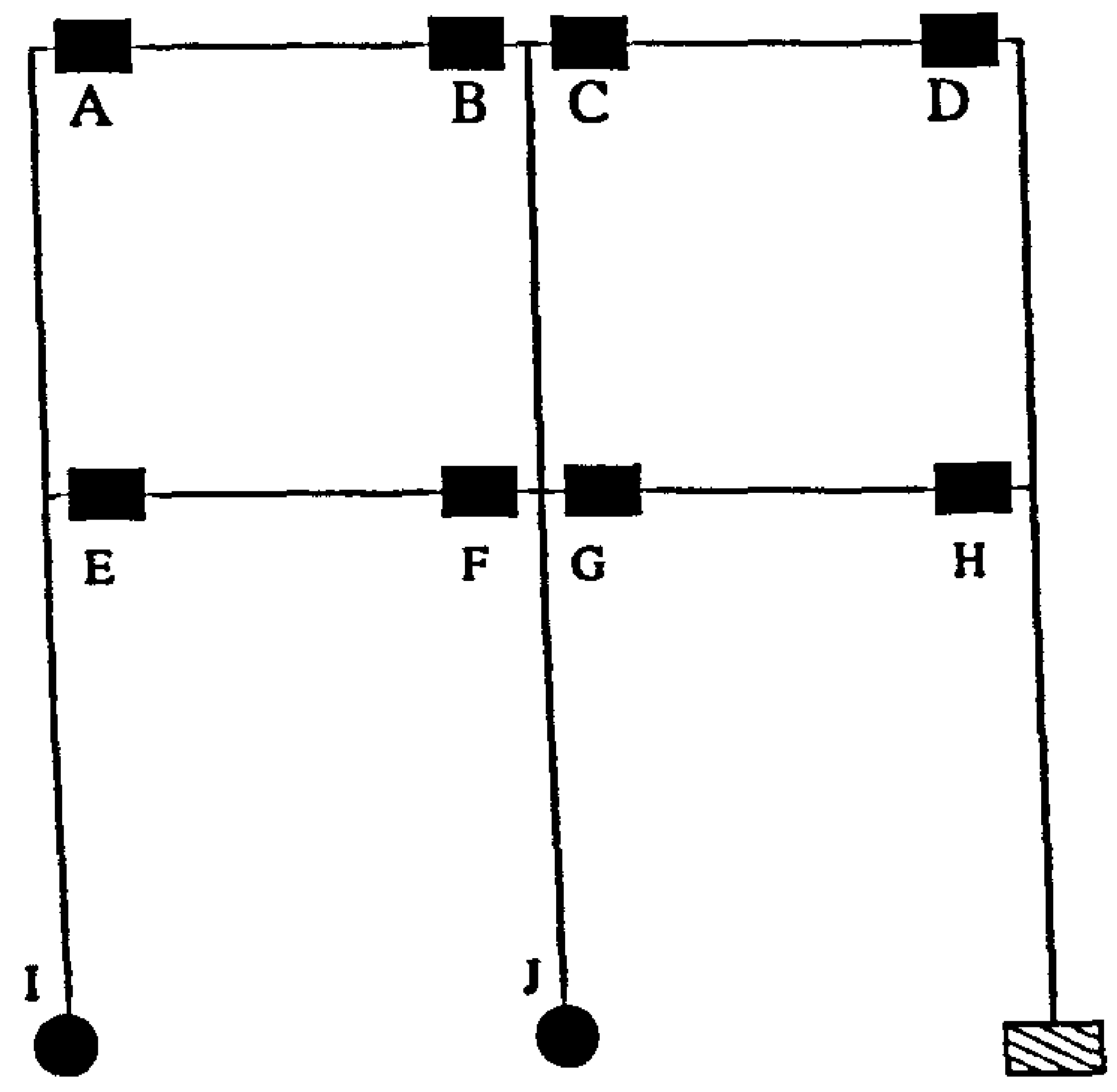
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.48	2.14
B	1.48	1.91
C	1.48	2.15
D	1.48	N/A
E	1.48	N/A
F	1.48	N/A
G	1.48	N/A
H	1.48	N/A
I	1.48	N/A

Key:

- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

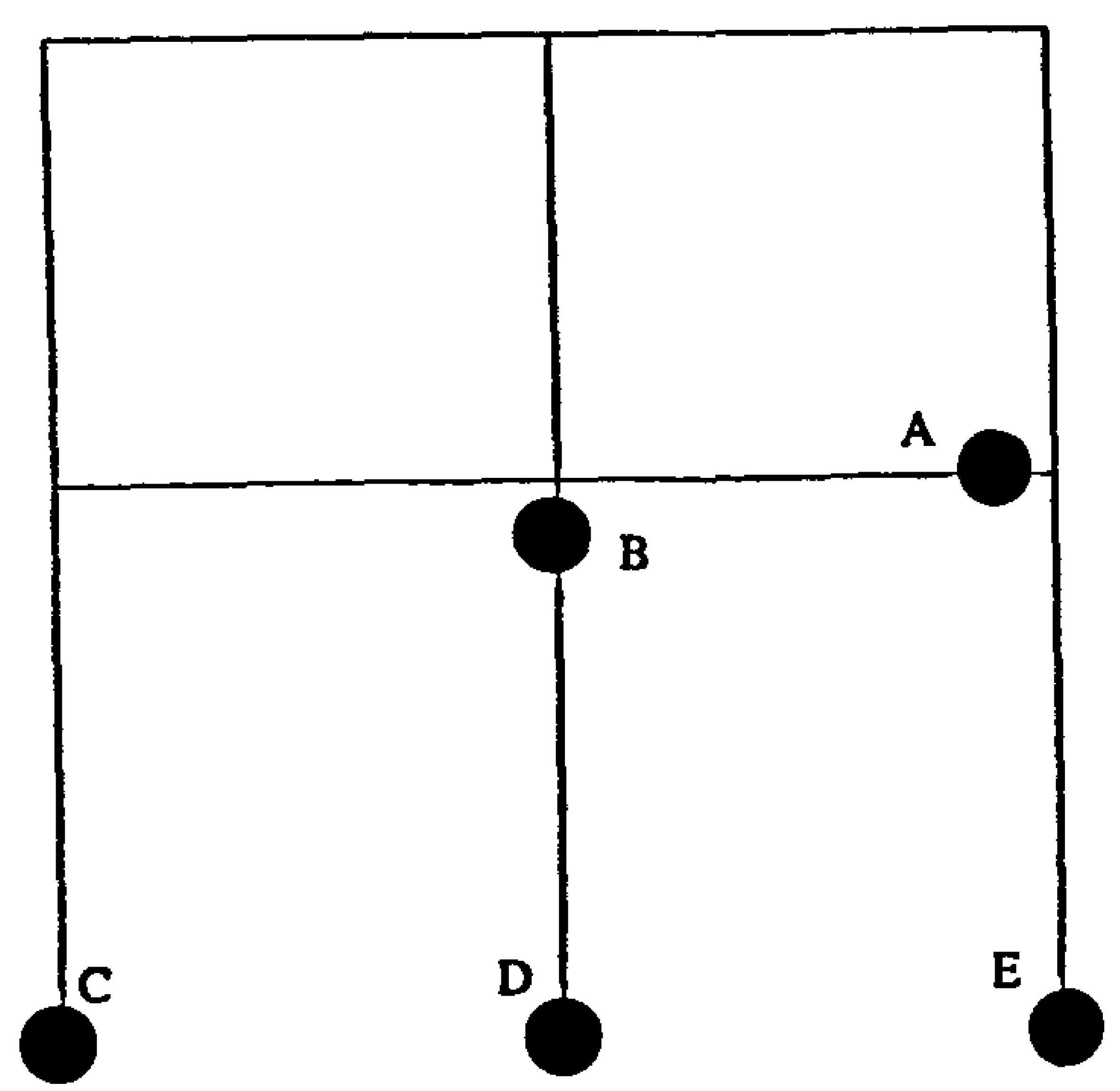
Frame 14
Load Case 2

Semi-Rigid Frame (Section Designation II)



Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.35	2.02
B	1.35	2.01
C	1.35	1.97
D	1.35	1.74
E	1.35	1.98
F	1.35	N/A
G	1.35	N/A
H	1.35	N/A
I	1.35	N/A
J	1.35	N/A

Rigid Frame (Section Designation II)



Key:

Semi-rigid connection

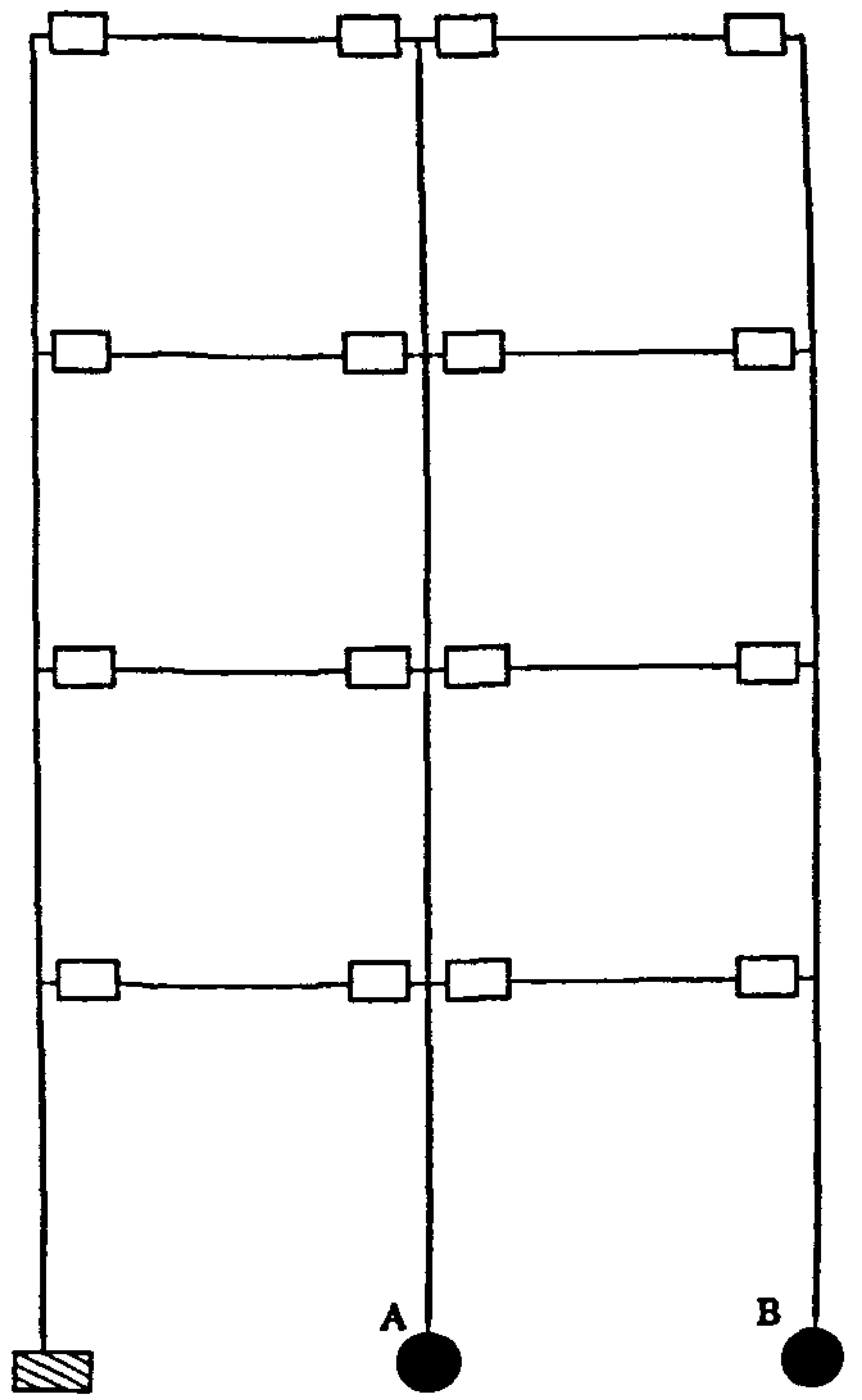
Plastification prior to ULS design load being attained

Plastification following the attainment of the design load

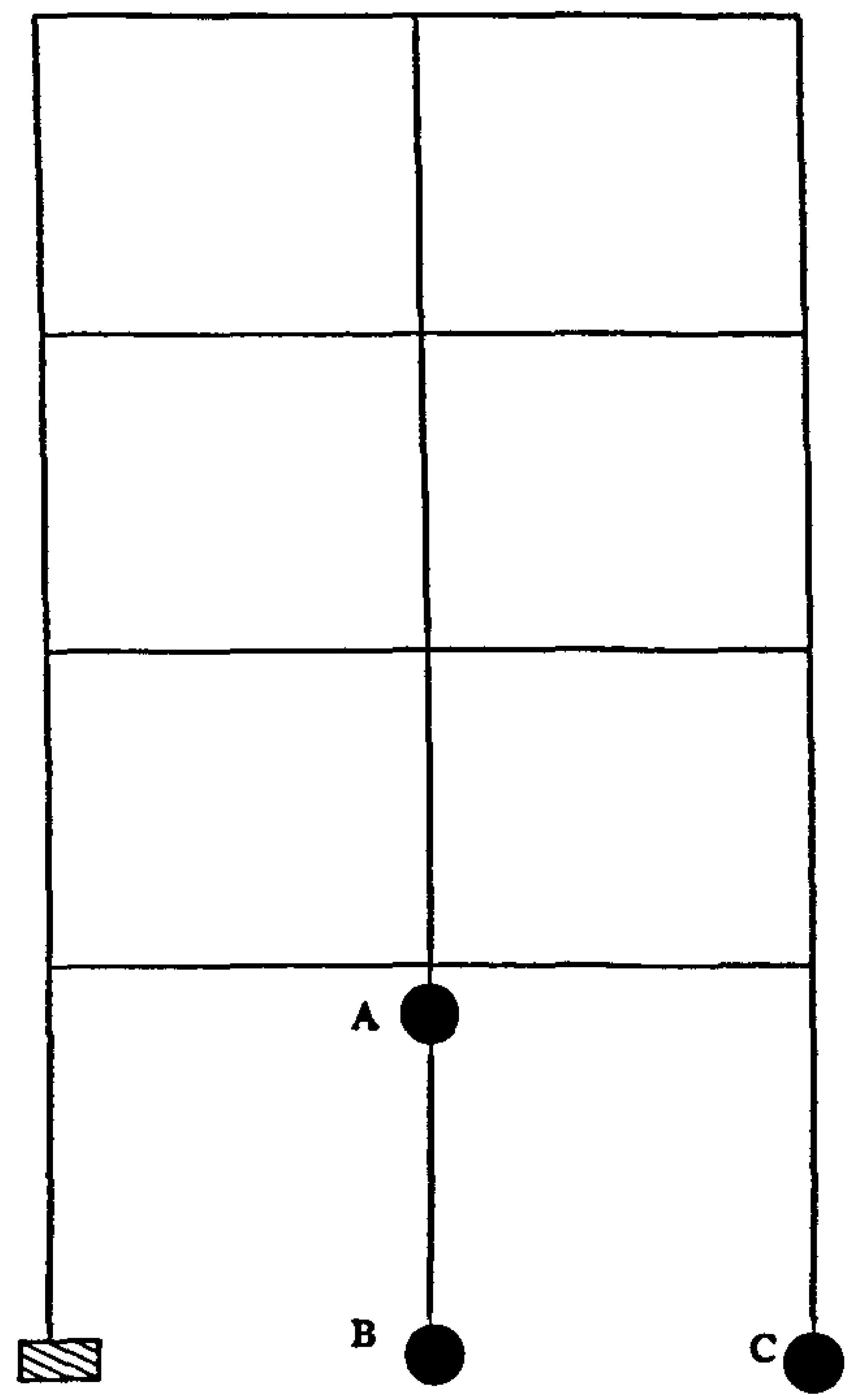
Member plastification

Frame 14
Load Case 3

Semi-Rigid Frame (Section Designation II)



Rigid Frame (Section Designation II)



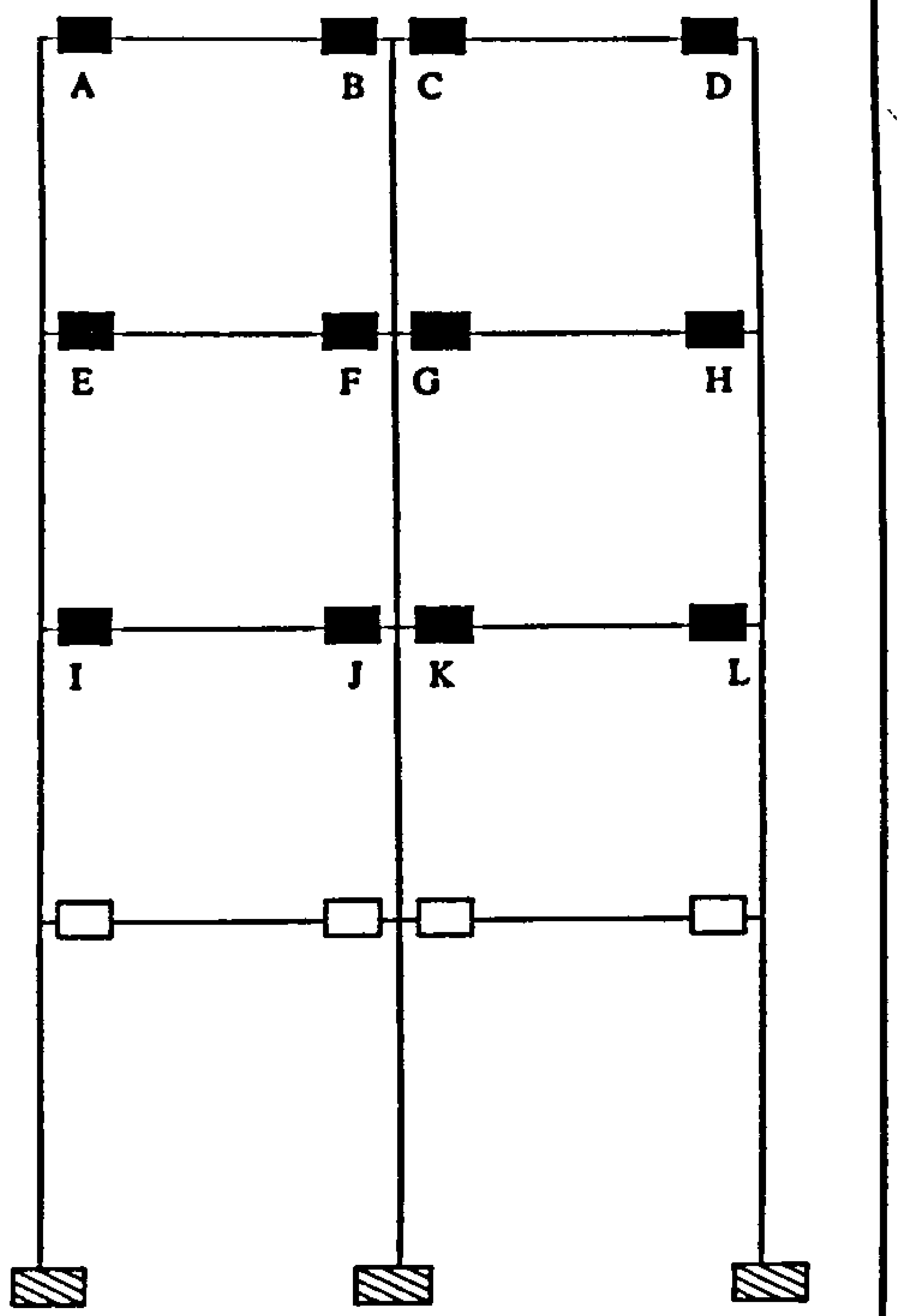
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	6.13	6.57
B	6.17	6.51
C	N/A	6.57

Key:

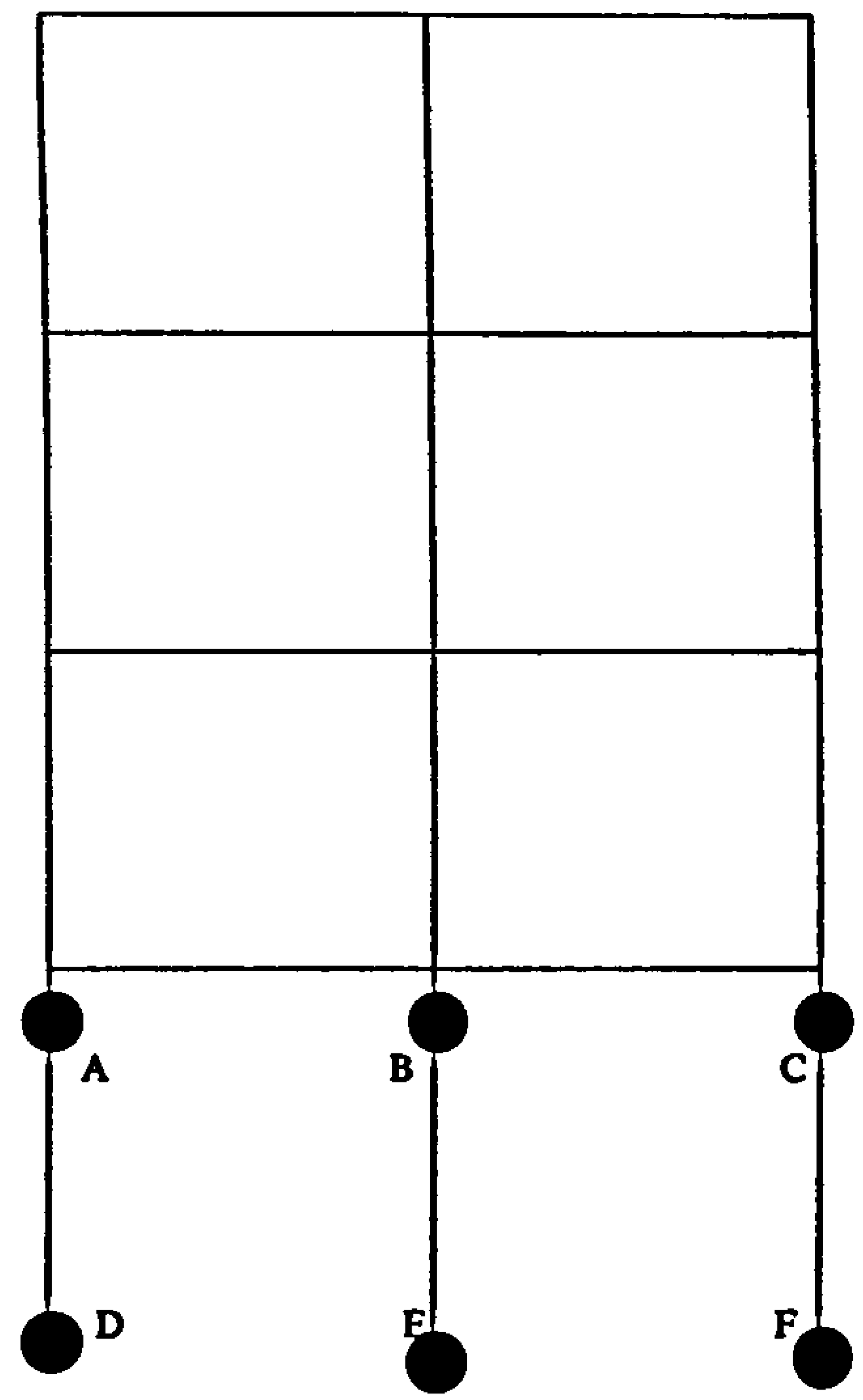
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 15
Load Case 1

Semi-Rigid Frame (Section Designation II)



Rigid Frame (Section Designation II)



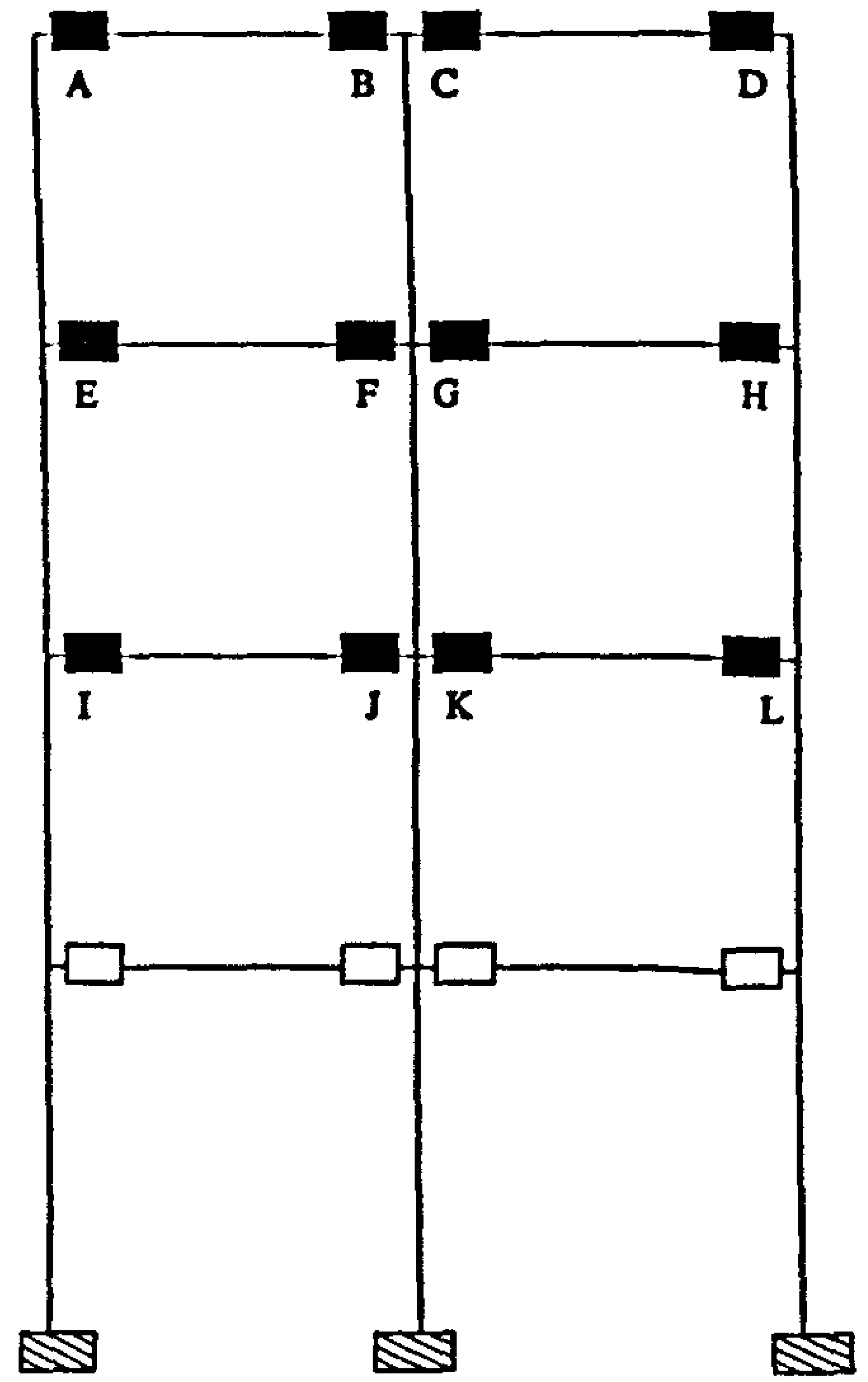
Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.36	2.02
B	1.36	2.02
C	1.36	2.02
D	1.36	1.96
E	1.36	1.90
F	1.36	1.97
G	1.36	N/A
H	1.36	N/A
I	1.36	N/A
J	1.36	N/A
K	1.36	N/A
L	1.36	N/A

Frame 15
Load Case 2

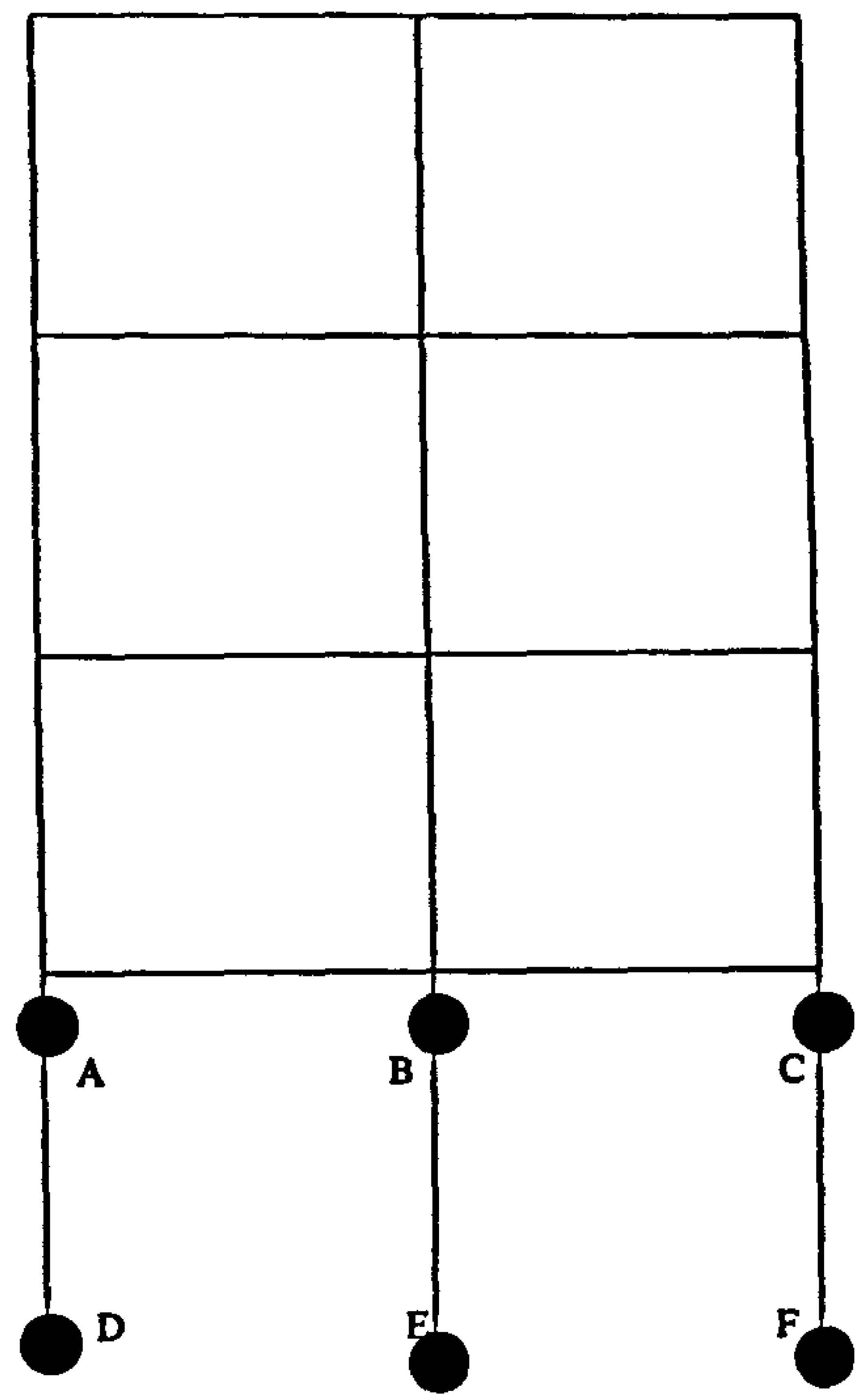
Key:

- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Semi-Rigid Frame (Section Designation II)



Rigid Frame (Section Designation II)



Frame 15
Load Case 3

Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.21	1.83
B	1.21	1.82
C	1.21	1.83
D	1.21	1.75
E	1.21	1.70
F	1.21	1.76
G	1.21	N/A
H	1.21	N/A
I	1.21	N/A
J	1.21	N/A
K	1.21	N/A
L	1.21	N/A

Key:

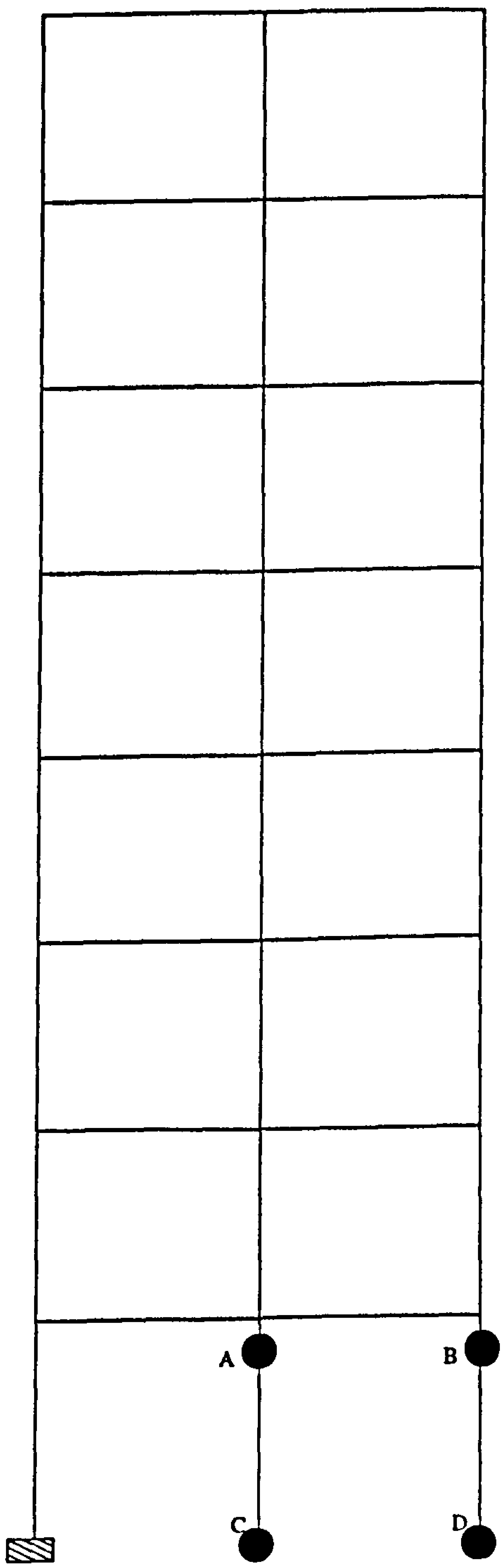
Semi-rigid connection

Plastification prior to ULS design load being attained

Plastification following the attainment of the design load

Member plastification

Rigid Frame
(Section Designation II)



Hinge Loaction	Load Level at Hinge Formation Rigid
A	6.33
B	6.48
C	6.28
D	6.48

Key:

Semi-rigid connection

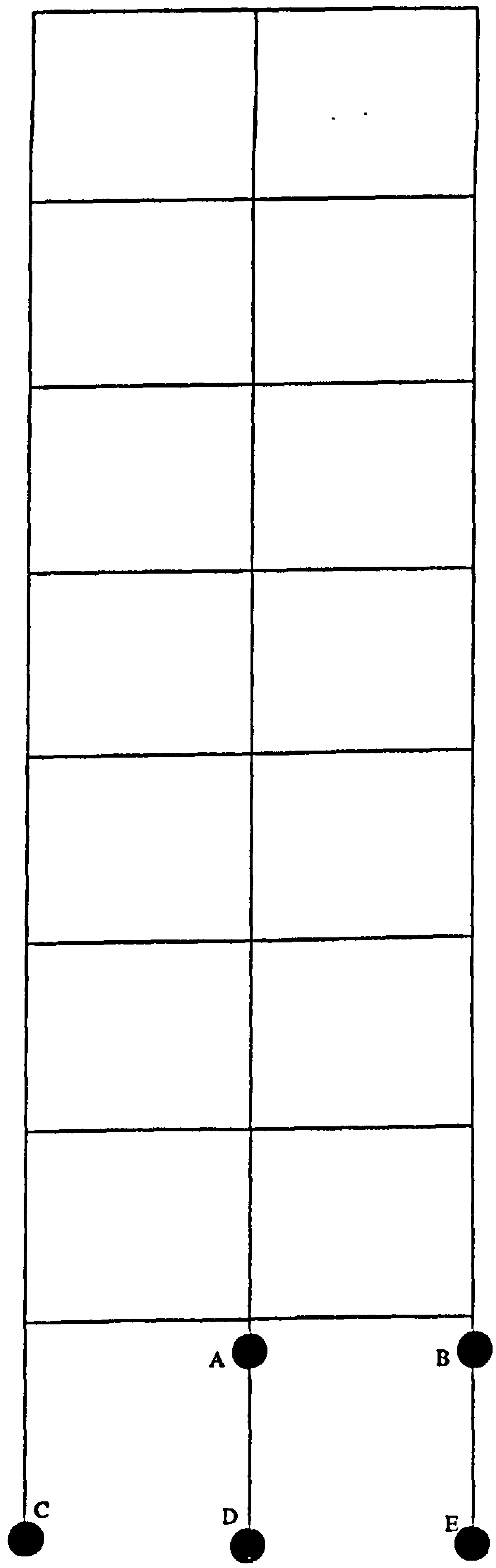
Plastification prior to ULS design load being attained

Plastification following the attainment of the design load

Member plastification

Frame 16
Load Case 1

Rigid Frame
(Section Designation II)

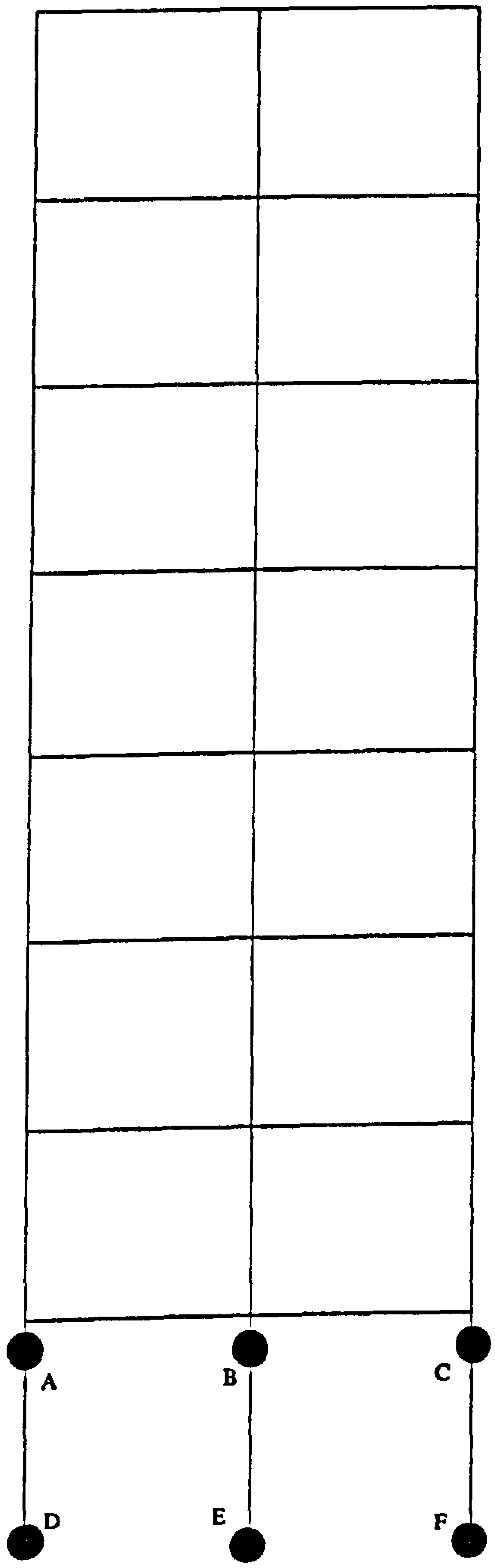


Hinge Loaction	Load Level at Hinge Formation Rigid
A	1.90
B	1.91
C	1.87
D	1.82
E	1.85

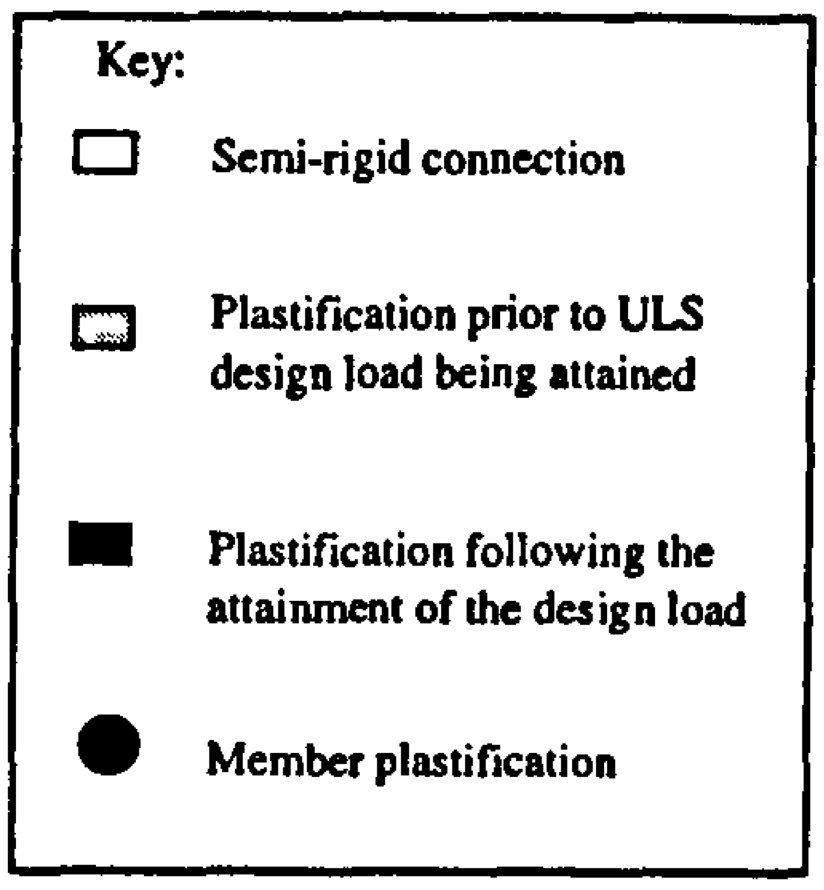
Key:
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 16
Load Case 2

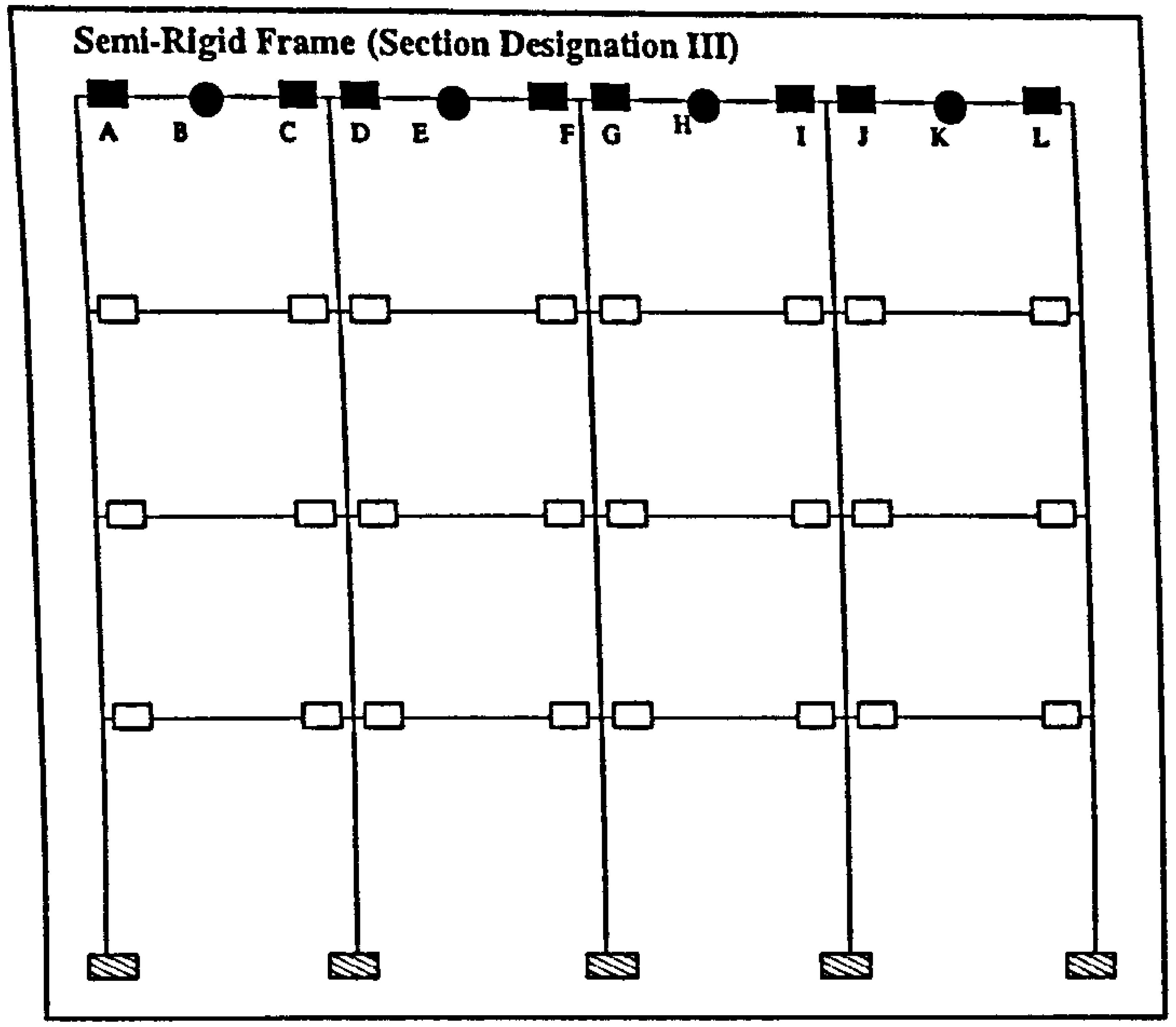
Rigid Frame
(Section Designation II)

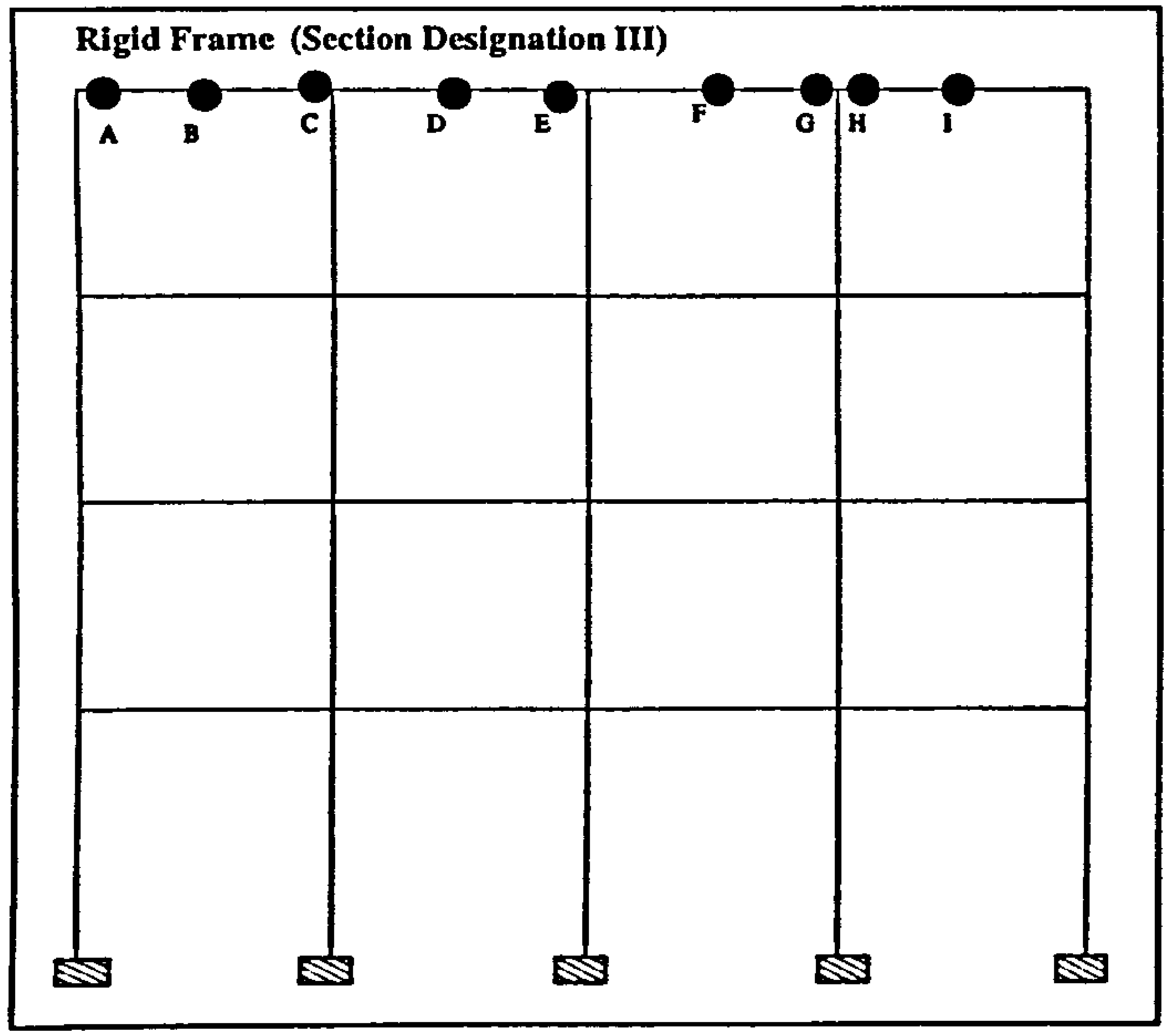


Hinge Loaction	Load Level at Hinge Formation Rigid
A	1.72
B	1.70
C	1.71
D	1.66
E	1.62
F	1.66

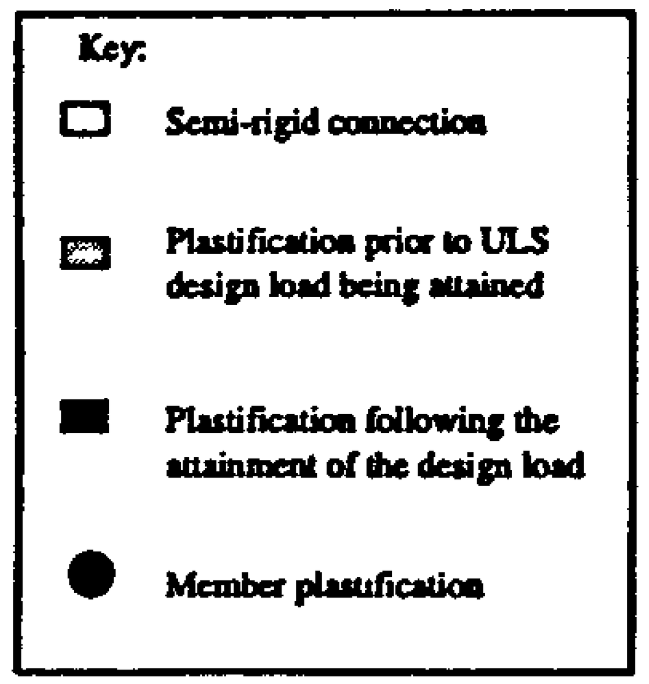


**Frame 16
Load Case 3**

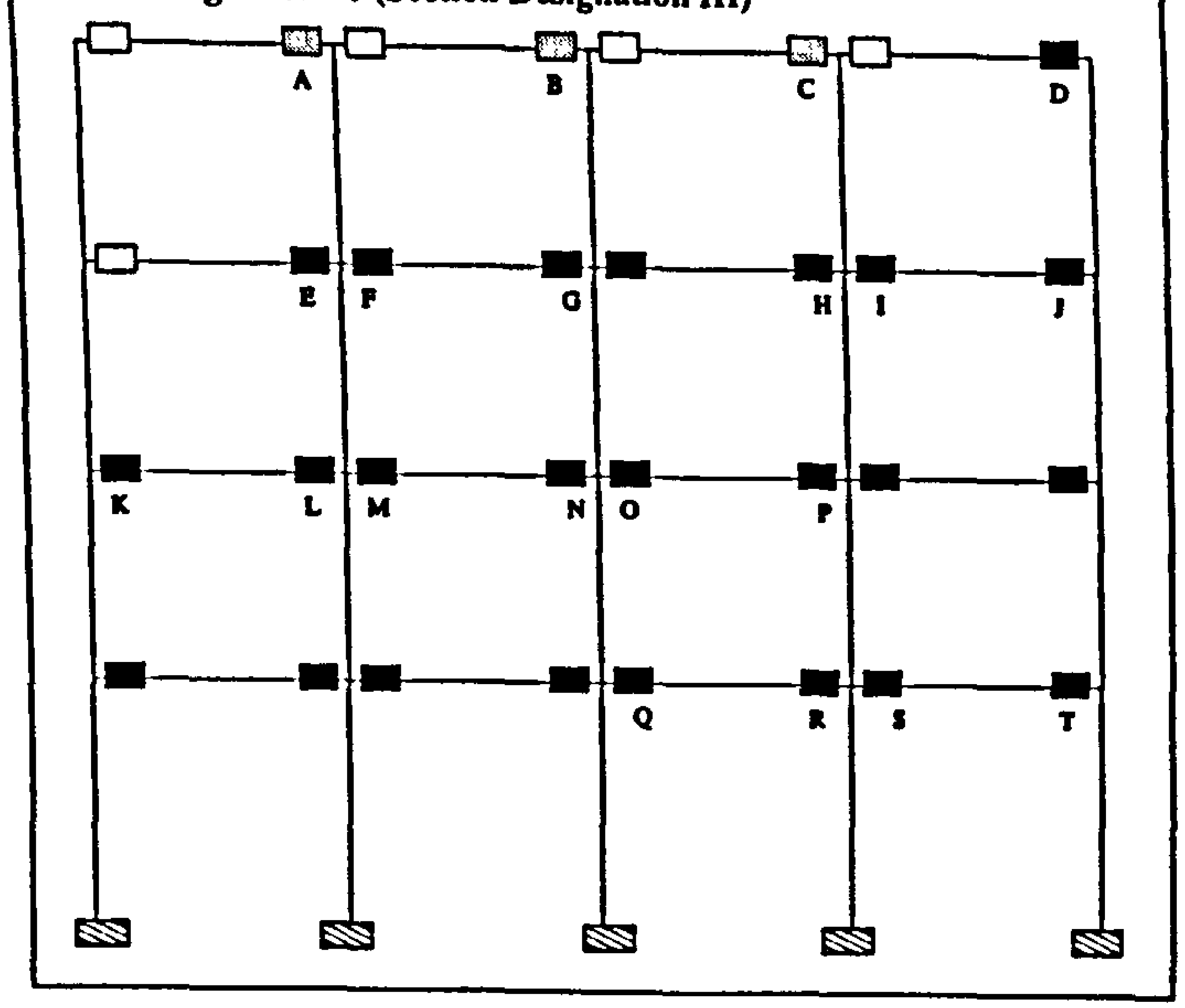




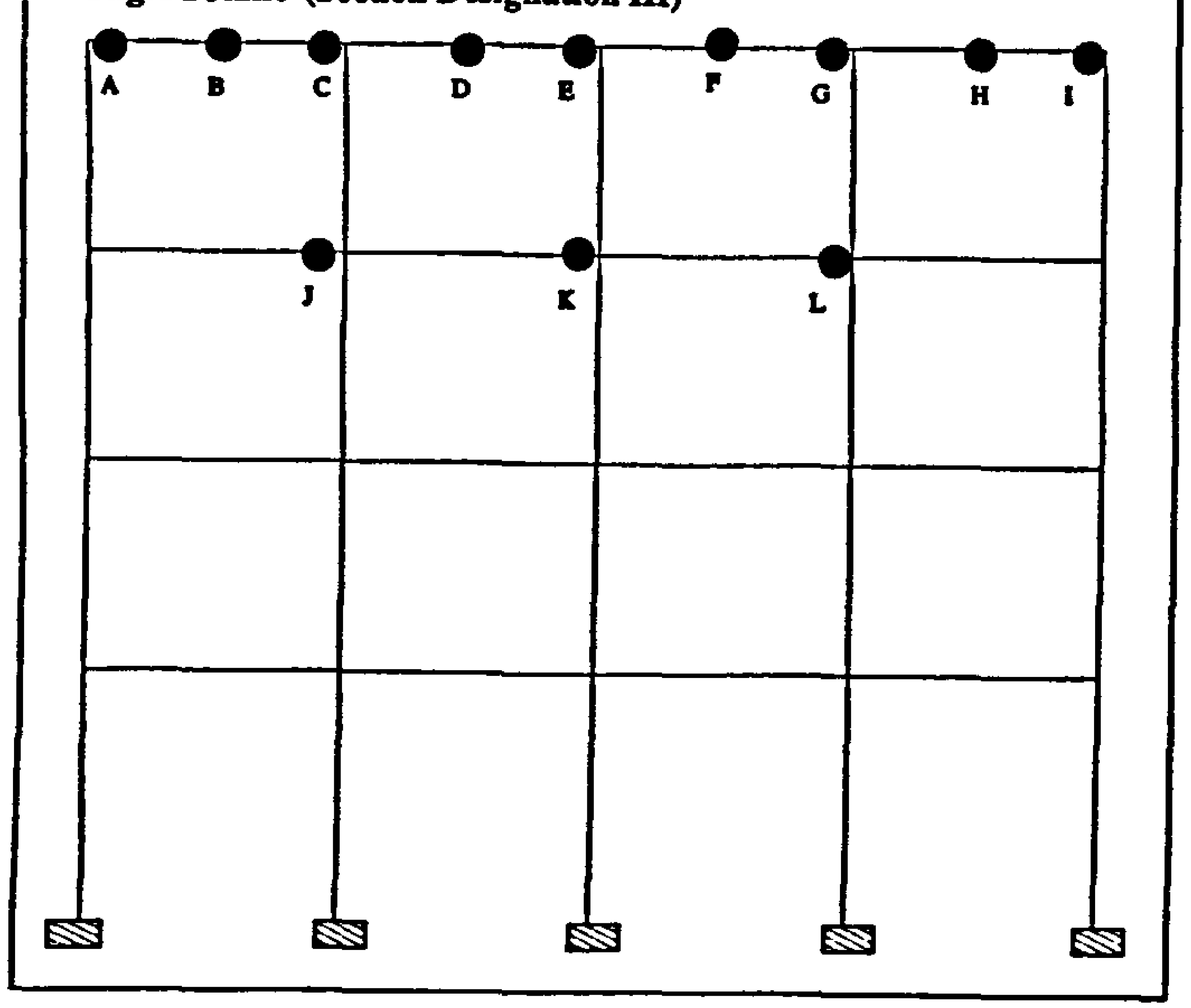
Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.31	2.03
B	1.27	1.78
C	1.27	1.73
D	1.31	2.01
E	1.30	1.98
F	1.27	2.02
G	1.31	1.98
H	1.30	1.80
I	1.27	1.80
J	1.31	N/A
K	1.30	N/A
L	1.27	N/A



Frame 17
Load Case 1



Semi-Rigid Frame (Section Designation III)



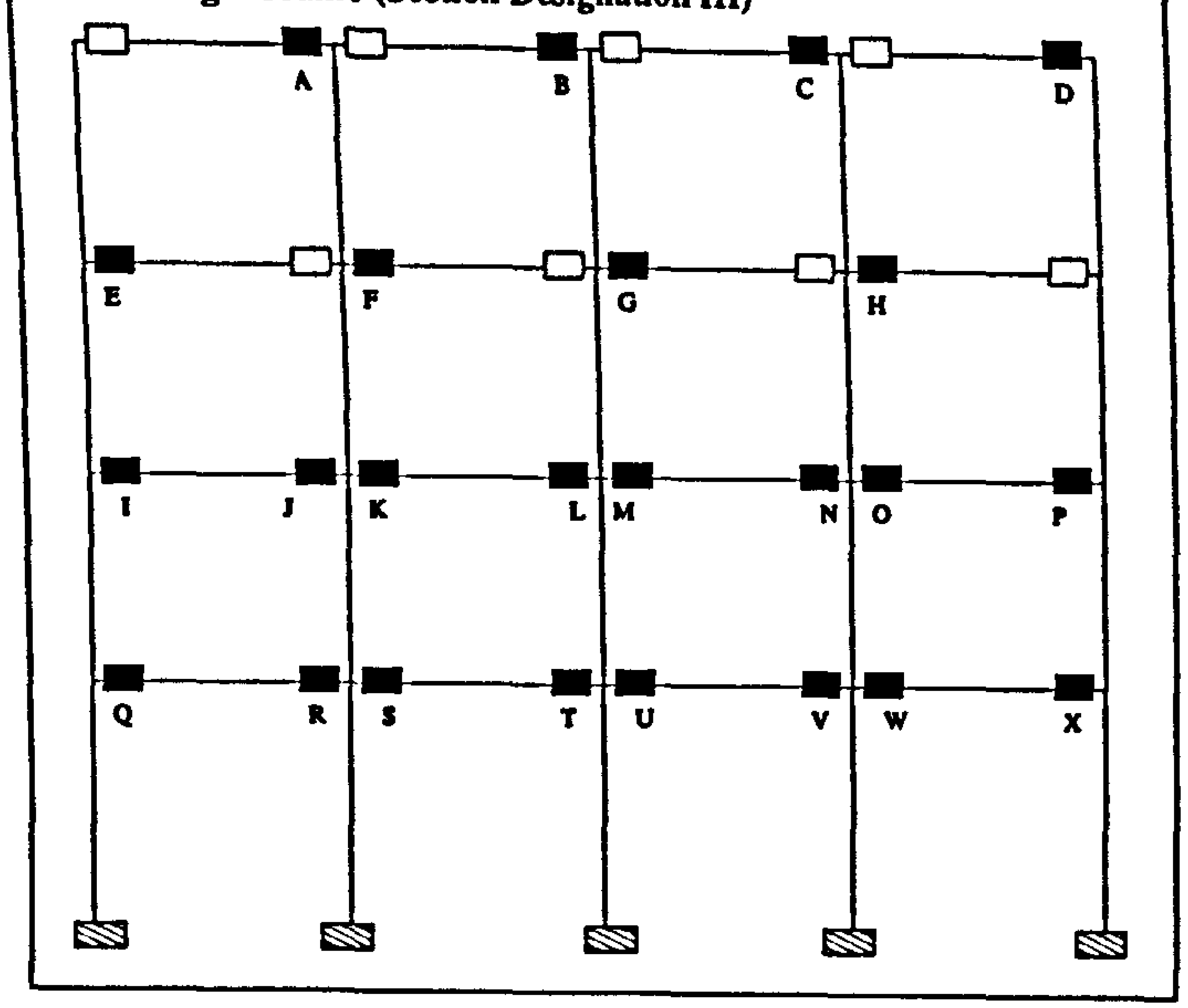
Rigid Frame (Section Designation III)

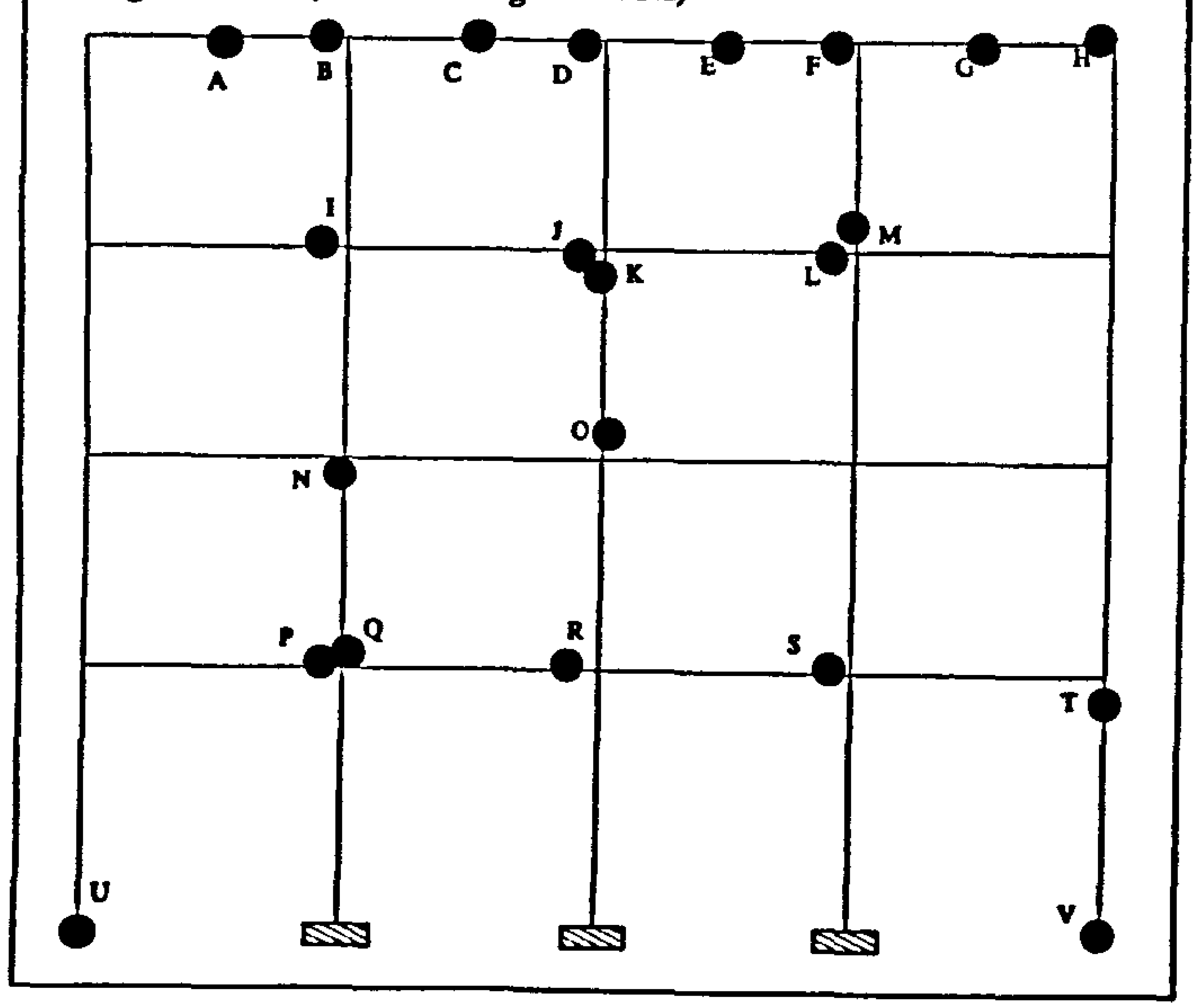
Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	< 1.00	2.36
B	< 1.00	2.03
C	< 1.00	1.80
D	1.38	2.22
E	1.38	2.01
F	1.38	2.23
G	1.38	1.96
H	1.38	2.14
I	1.38	2.29
J	1.38	2.13
K	1.38	2.23
L	1.38	2.21
M	1.38	N/A
N	1.38	N/A
O	1.38	N/A
P	1.38	N/A
Q	1.38	N/A
R	1.38	N/A
S	1.38	N/A
T	1.38	N/A
U	1.38	N/A
V	1.38	N/A
W	1.38	N/A
X	1.38	N/A
Y	1.38	N/A
Z	1.38	N/A
AA	1.38	N/A

Key:

- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 17
Load Case 2

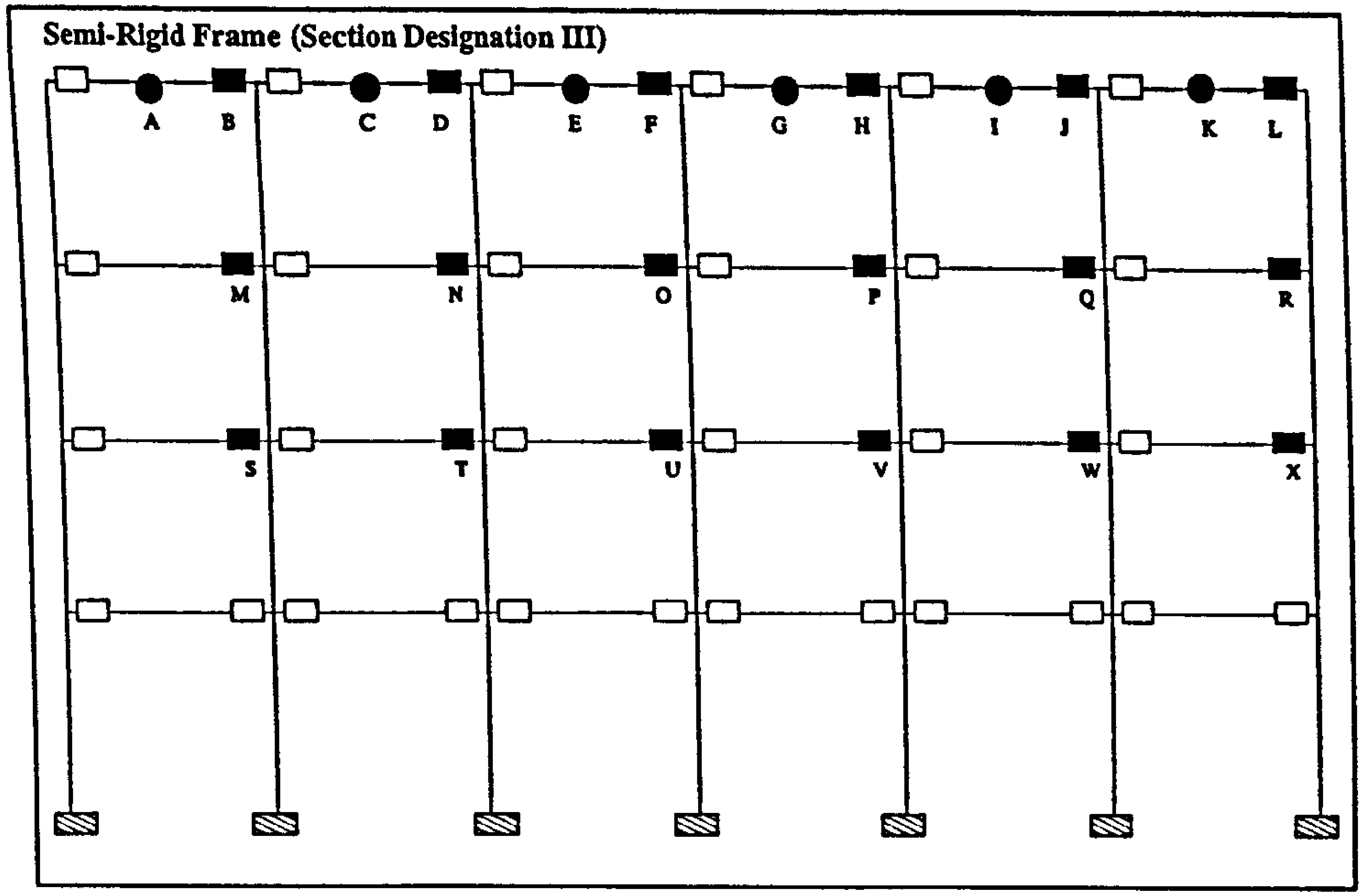


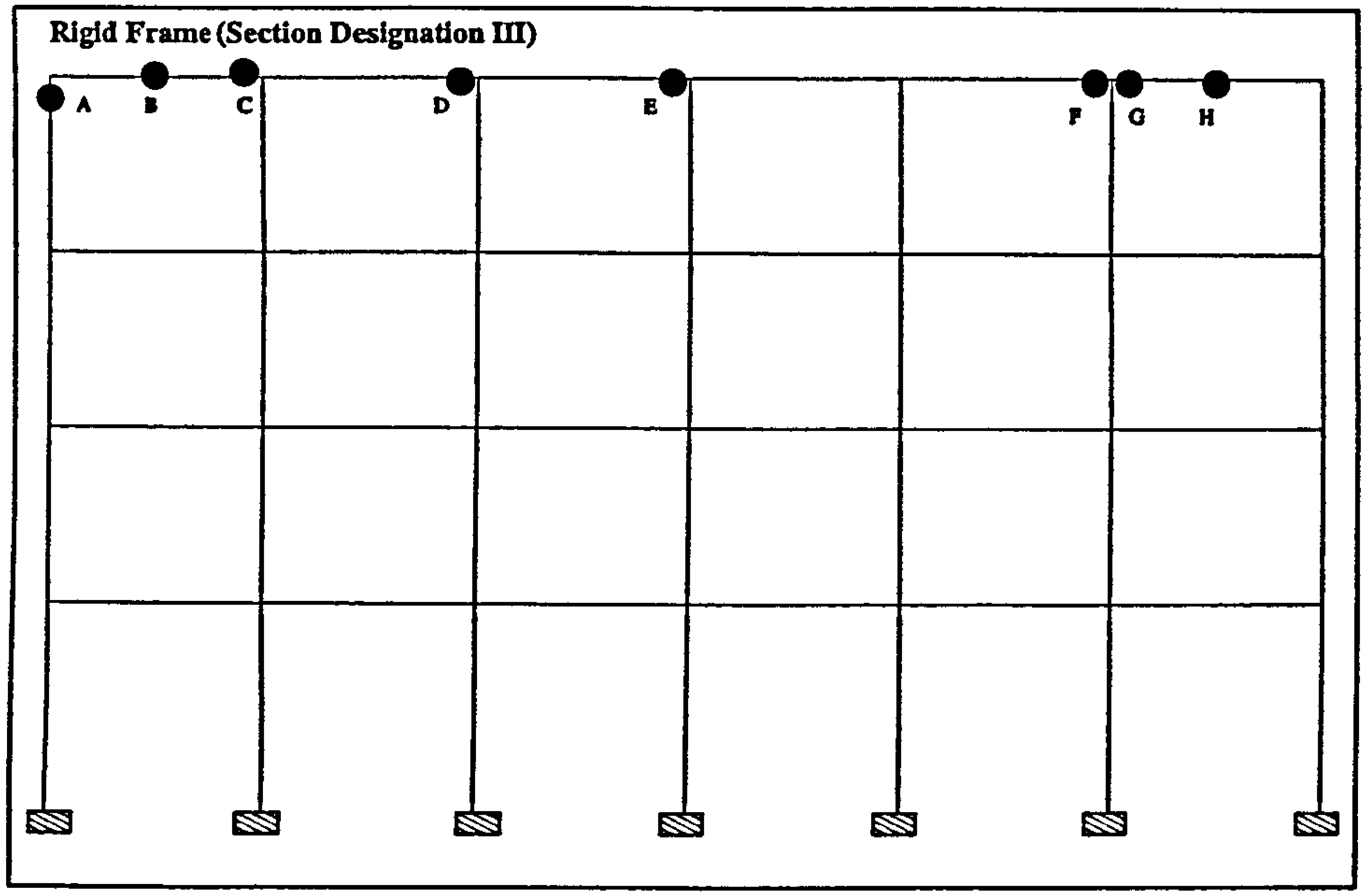


Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.28	2.35
B	1.28	2.02
C	1.28	2.58
D	1.28	2.25
E	1.28	2.60
F	1.28	2.19
G	1.28	2.52
H	1.28	2.65
I	1.28	2.70
J	1.28	2.72
K	1.28	2.74
L	1.28	2.71
M	1.28	2.71
N	1.28	2.74
O	1.28	2.62
P	1.28	2.63
Q	1.28	2.63
R	1.28	2.70
S	1.28	2.66
T	1.28	2.72
U	1.28	2.65
V	1.28	2.66
W	1.28	N/A
X	1.28	N/A

Key:
- ☐ Semi-rigid connection
- ▨ Plastification prior to ULS design load being attained
- ■ Plastification following the attainment of the design load
- ● Member plastification

Frame 17
Load Case 3

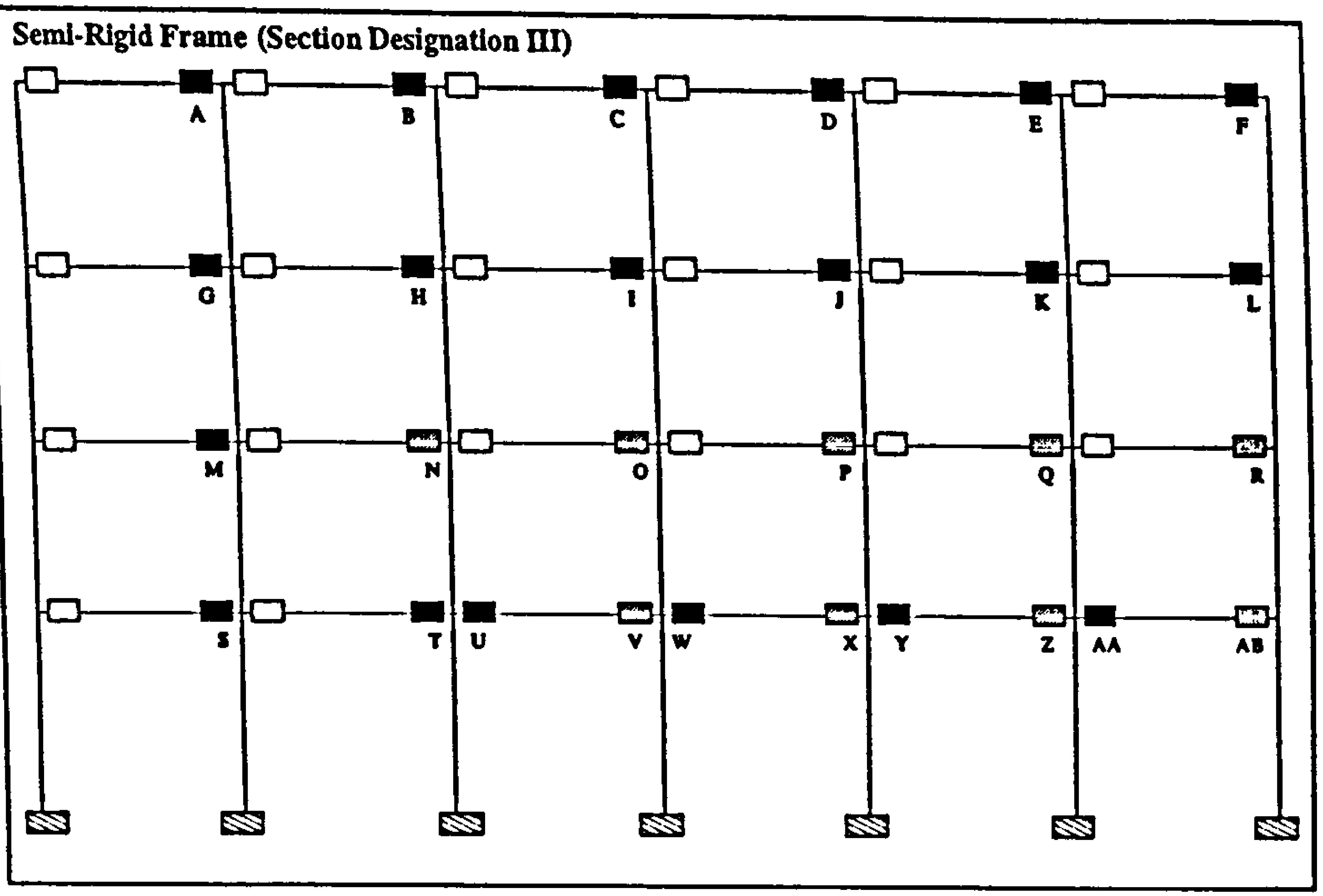


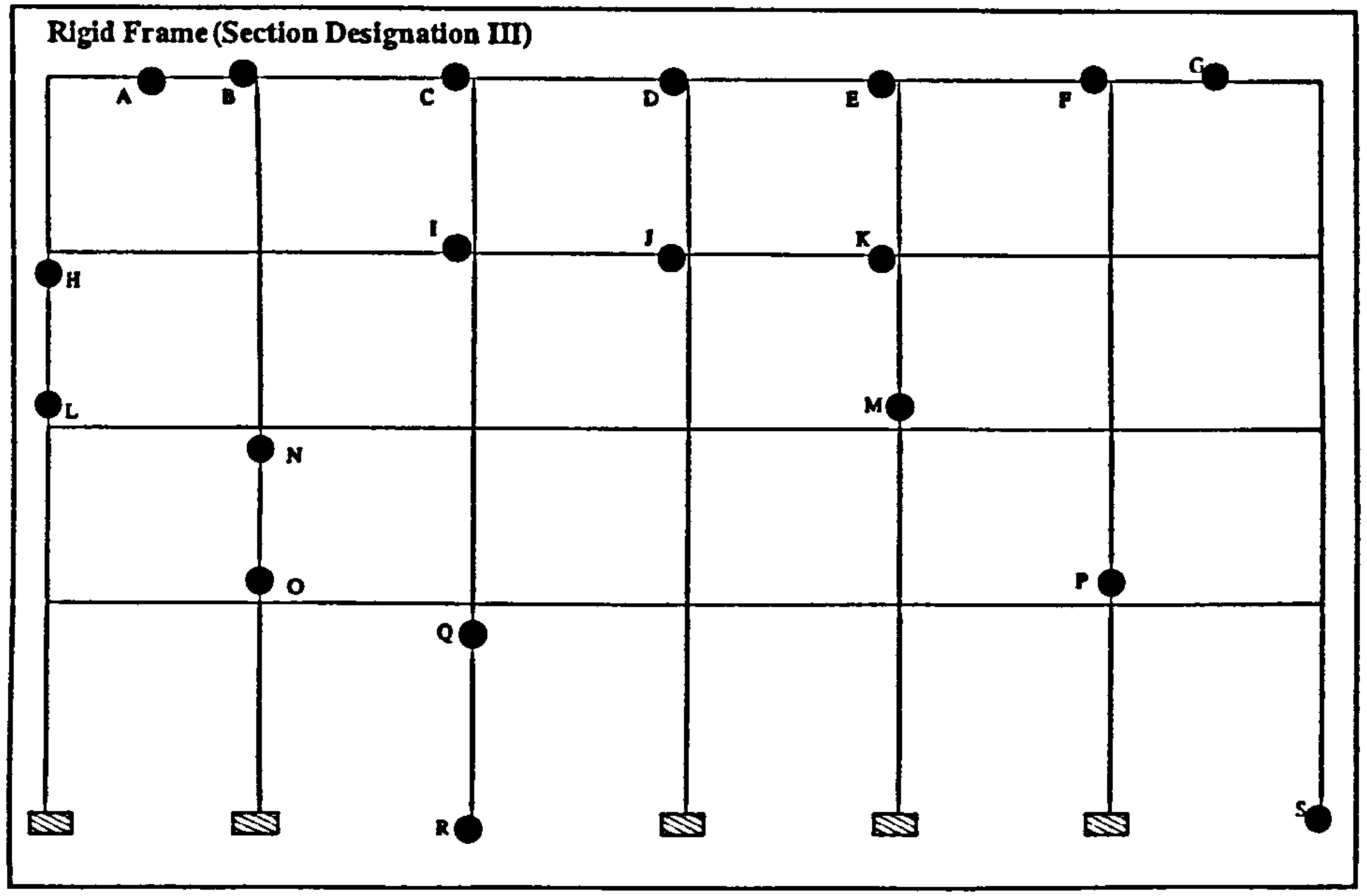


Hinge Location	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.37	2.00
B	1.43	1.75
C	1.43	1.78
D	1.43	1.95
E	1.43	2.00
F	1.43	1.99
G	1.43	1.89
H	1.43	1.85
I	1.43	N/A
J	1.43	N/A
K	1.43	N/A
L	1.43	N/A
M	1.43	N/A
N	1.43	N/A
O	1.43	N/A
P	1.43	N/A
Q	1.43	N/A
R	1.43	N/A
S	1.43	N/A
T	1.43	N/A
U	1.43	N/A
V	1.43	N/A
W	1.43	N/A
X	1.43	N/A

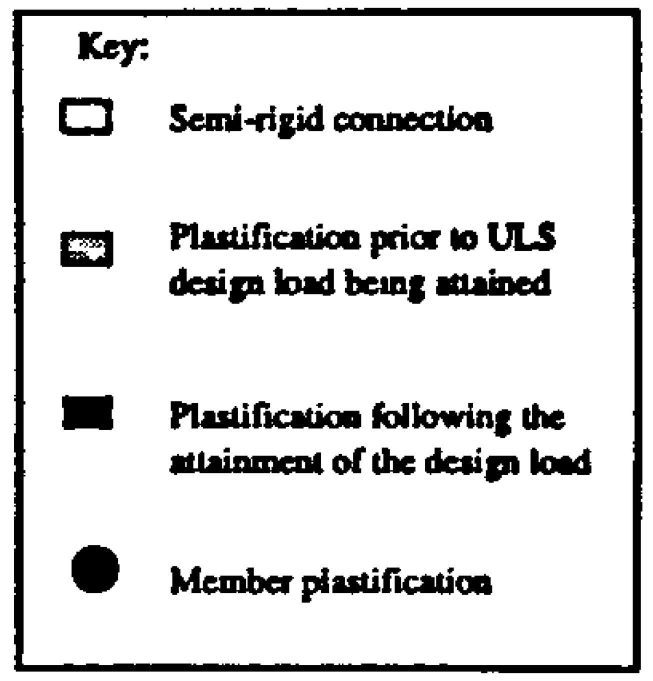
Key:
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 18
Load Case 1



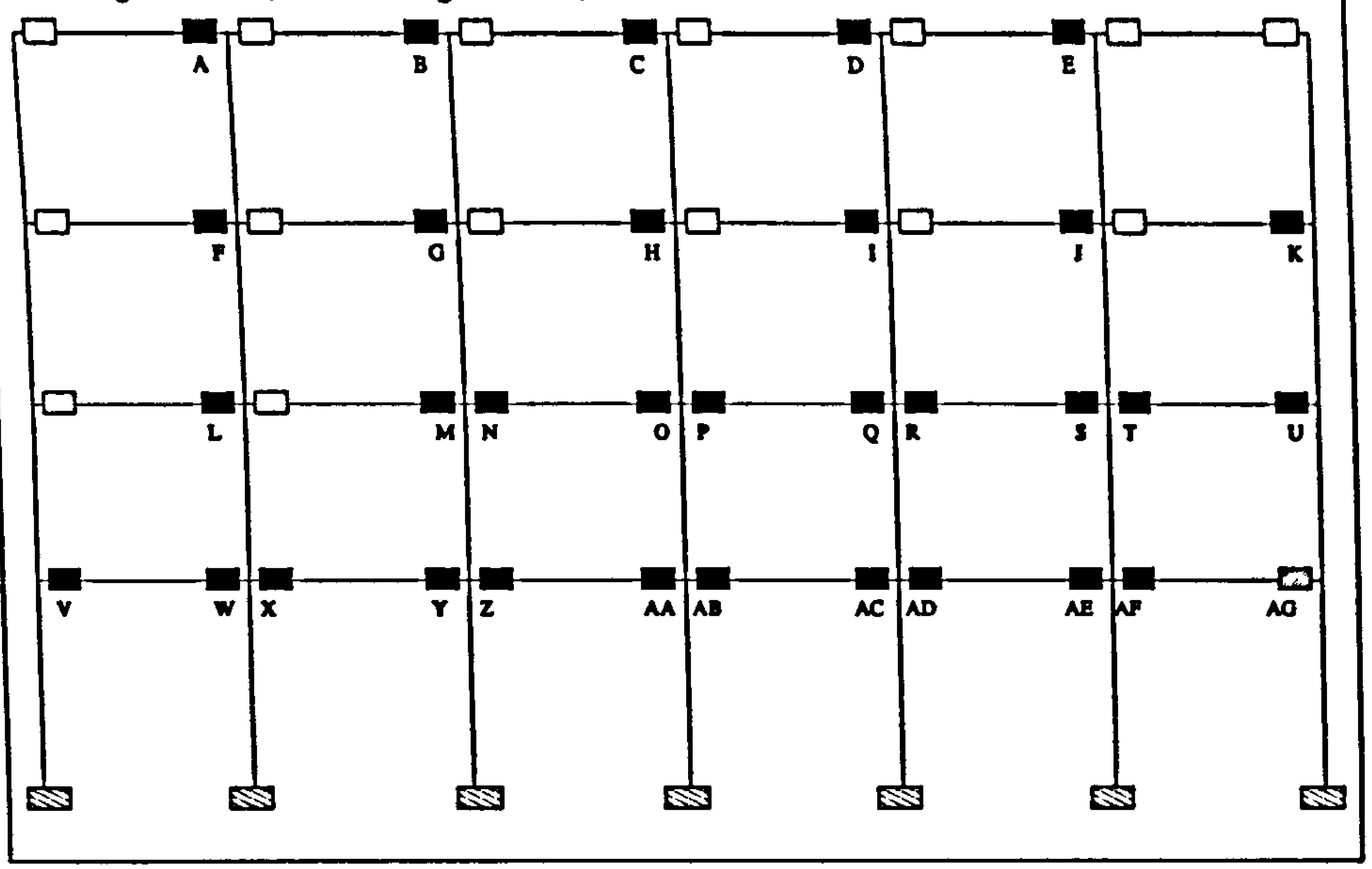


Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.31	2.06
B	1.31	2.01
C	1.31	2.13
D	1.31	2.15
E	1.31	2.16
F	1.31	2.13
G	1.31	2.24
H	1.31	2.25
I	1.31	2.26
J	1.31	2.28
K	1.31	2.28
L	1.31	2.23
M	1.31	2.25
N	< 1.00	2.28
O	< 1.00	2.24
P	< 1.00	2.24
Q	< 1.00	2.28
R	< 1.00	2.24
S	1.31	2.27
T	1.31	N/A
U	1.31	N/A
V	< 1.00	N/A
W	1.31	N/A
X	< 1.00	N/A
Y	1.31	N/A
Z	< 1.00	N/A
AA	1.31	N/A
AB	< 1.00	N/A

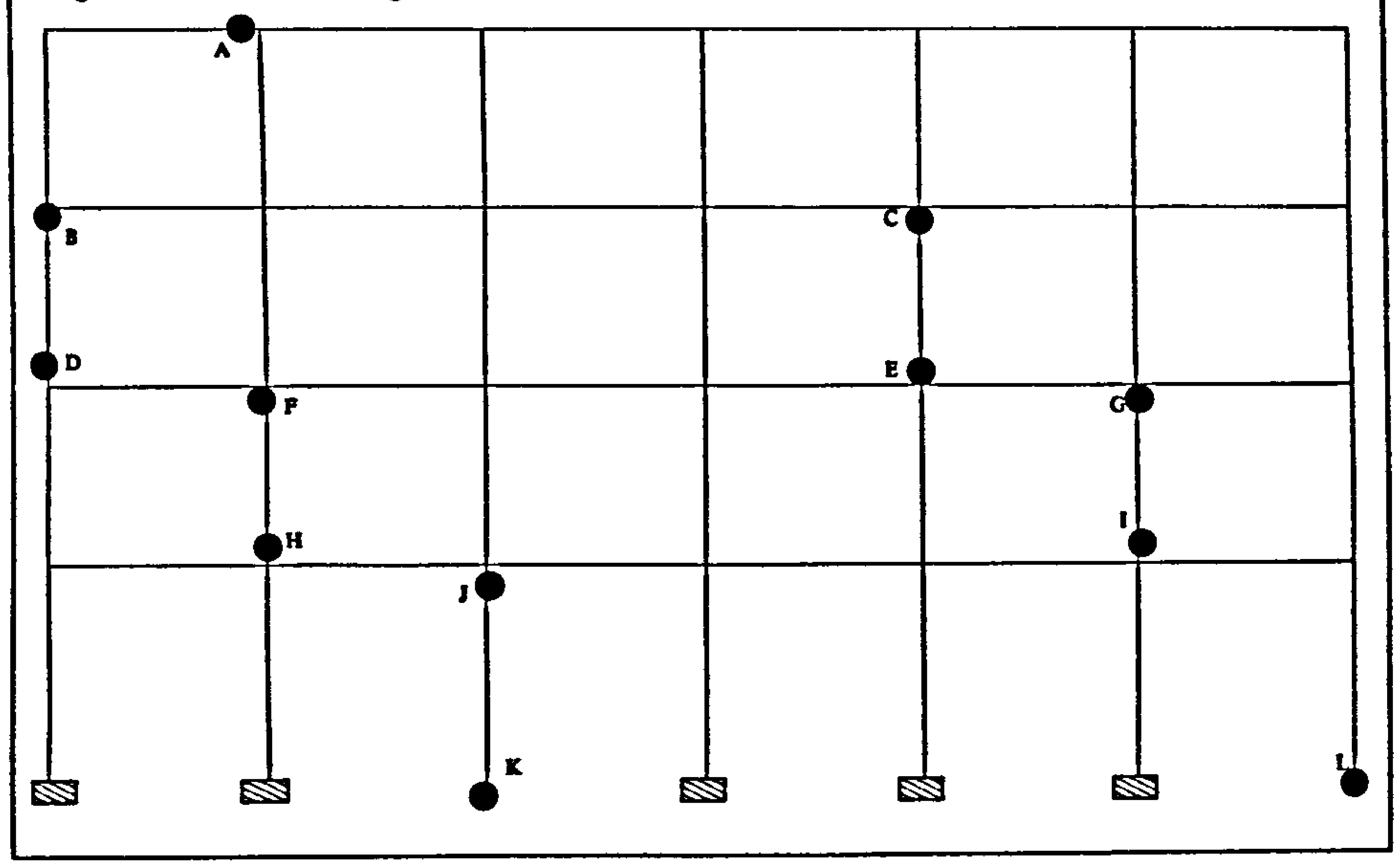


Frame 18
Load Case 2

Semi-Rigid Frame (Section Designation III)



Rigid Frame (Section Designation III)

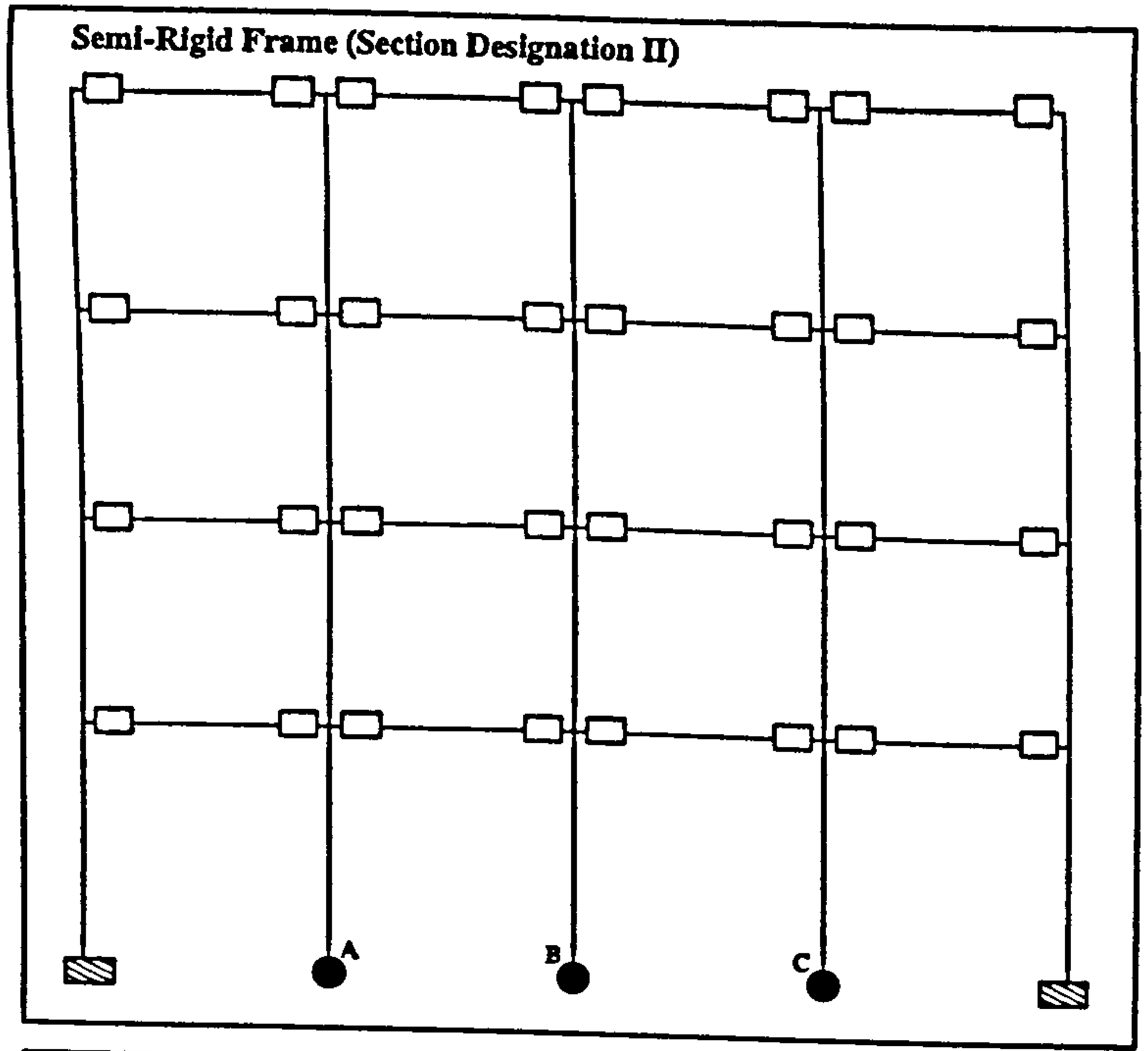


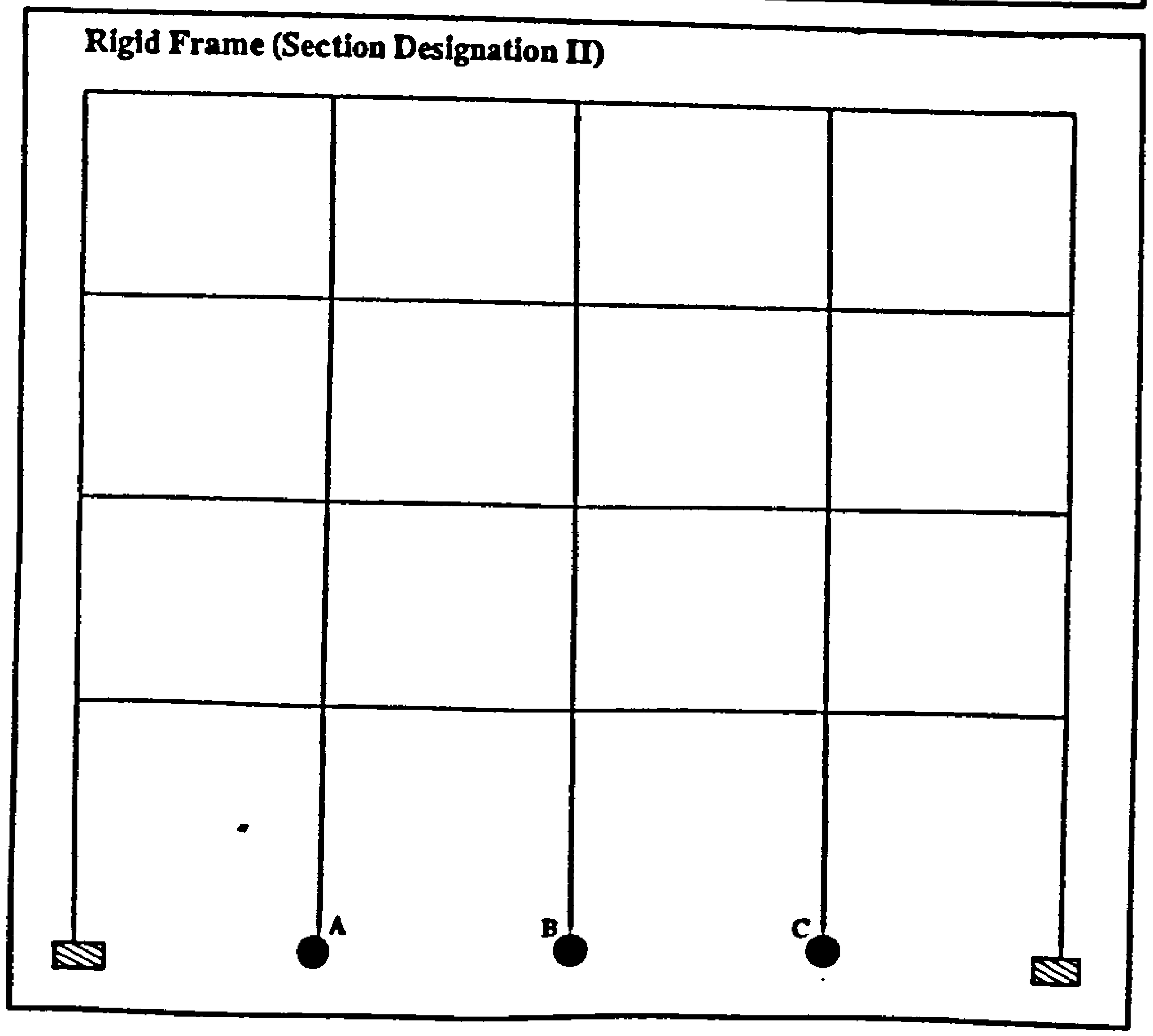
Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.42	2.29
B	1.42	2.32
C	1.42	2.33
D	1.42	2.31
E	1.42	2.26
F	1.42	2.32
G	1.42	2.33
H	1.42	2.24
I	1.42	2.25
J	1.42	2.32
K	1.42	2.24
L	1.42	2.29
M	1.42	N/A
N	1.42	N/A
O	1.42	N/A
P	1.42	N/A
Q	1.42	N/A
R	1.42	N/A
S	1.42	N/A
T	1.42	N/A
U	1.42	N/A
V	1.42	N/A
W	1.42	N/A
X	1.42	N/A
Y	1.42	N/A
Z	1.42	N/A
AA	1.42	N/A
AB	1.42	N/A
AC	1.42	N/A
AD	1.42	N/A
AE	1.42	N/A
AF	< 1.00	N/A

Key:

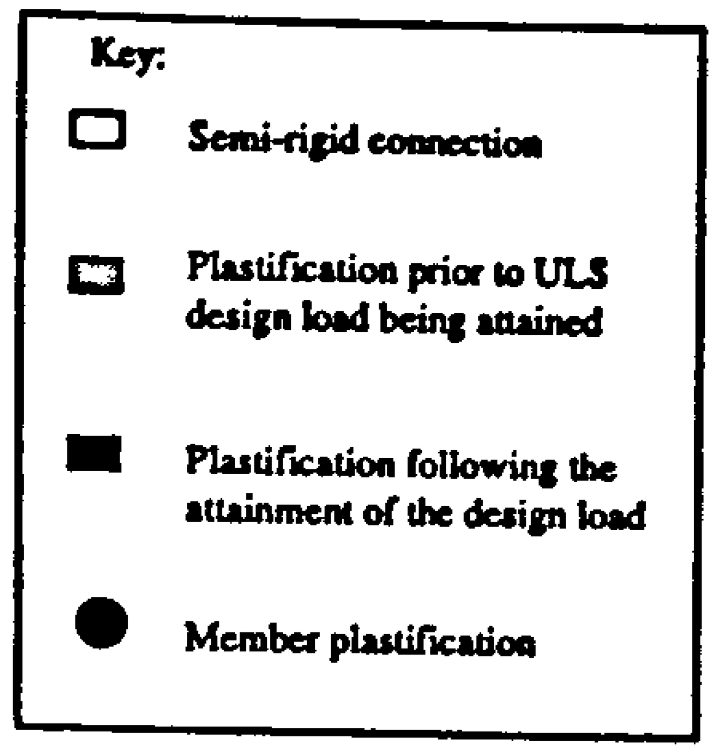
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 18
Load Case 3



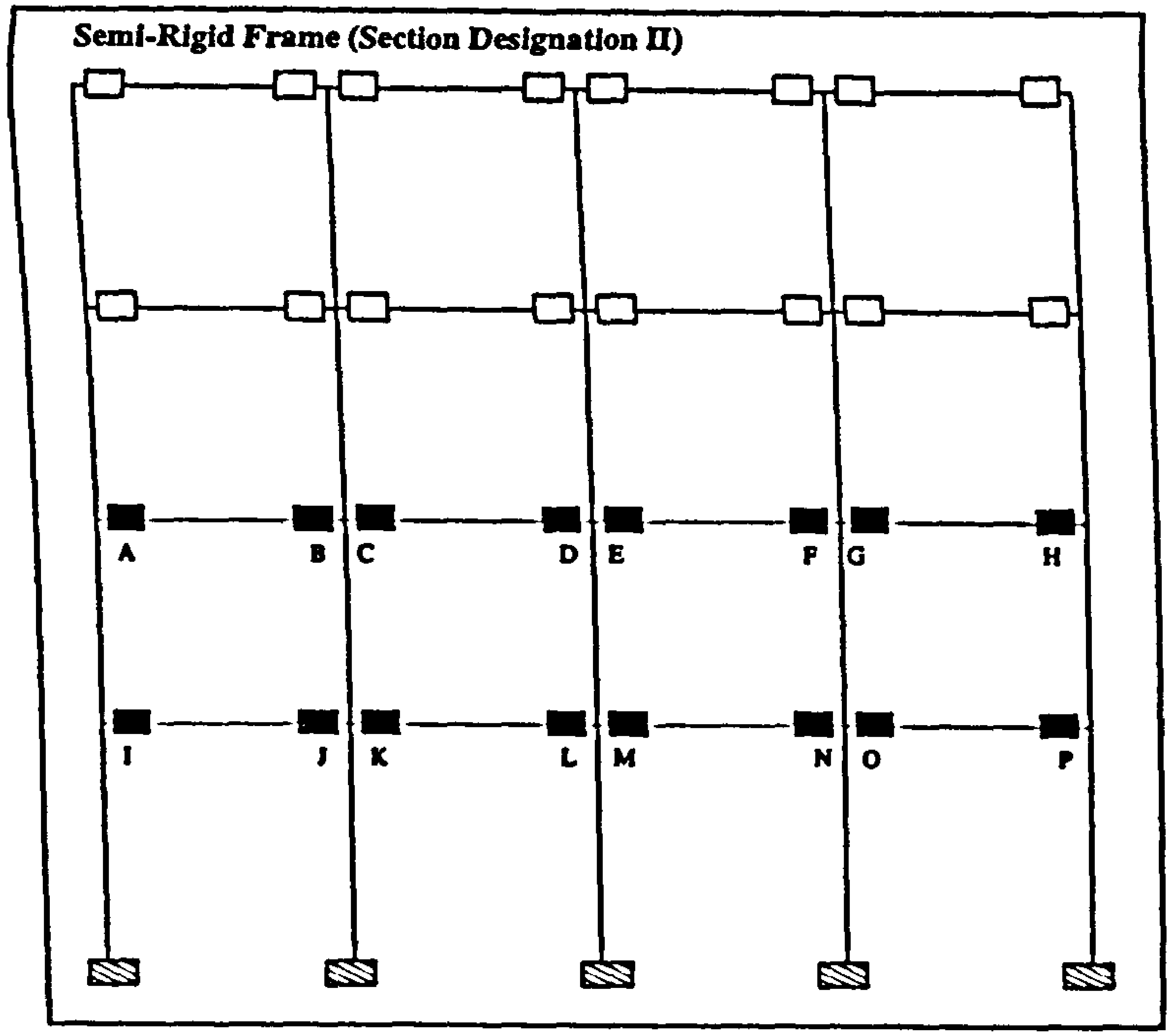


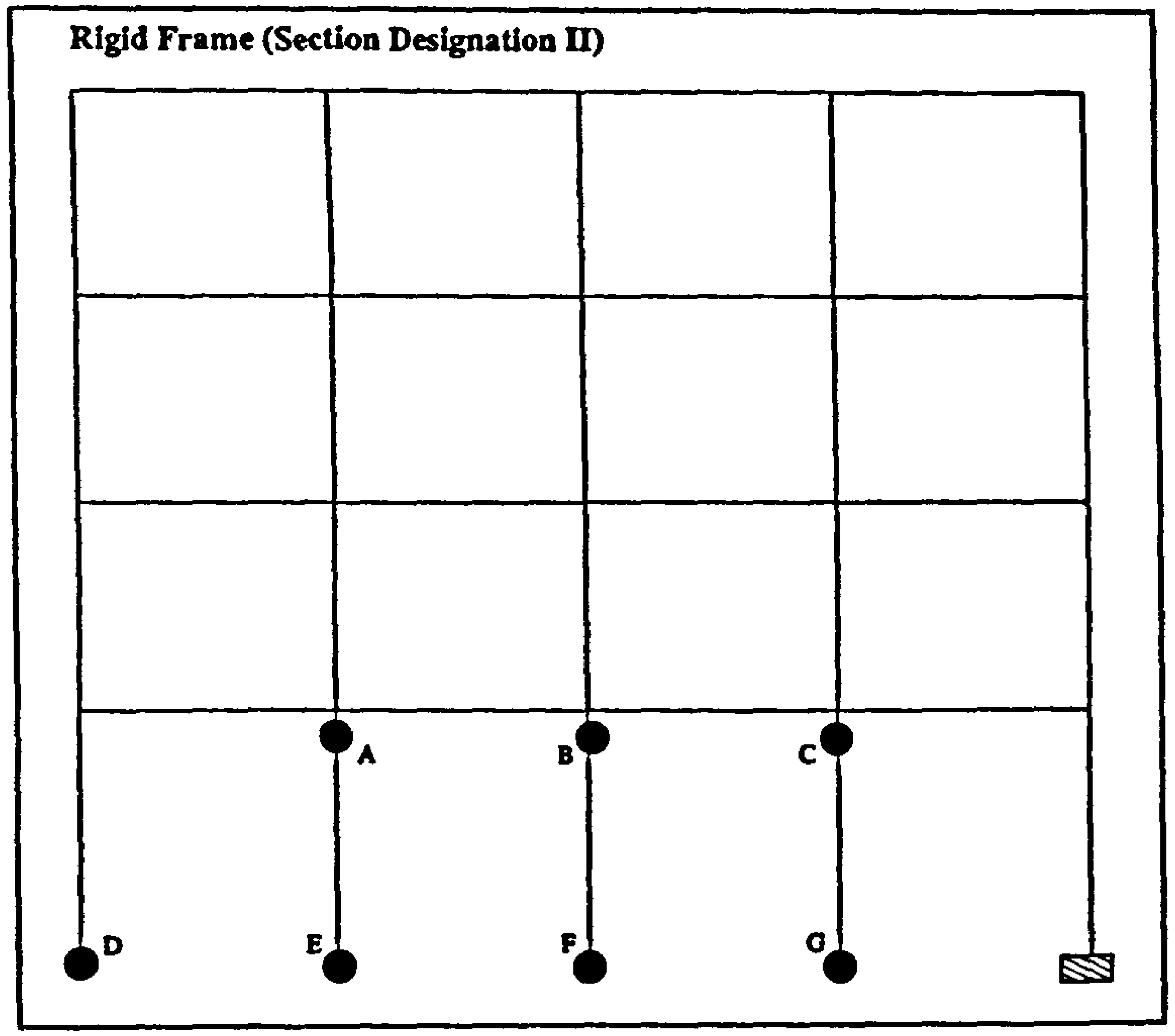
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	3.05	3.43
B	3.05	3.42
C	3.05	3.42



Frame 19
Load Case 1

Plastic-hinges in minor -axis "wind-moment" frames.





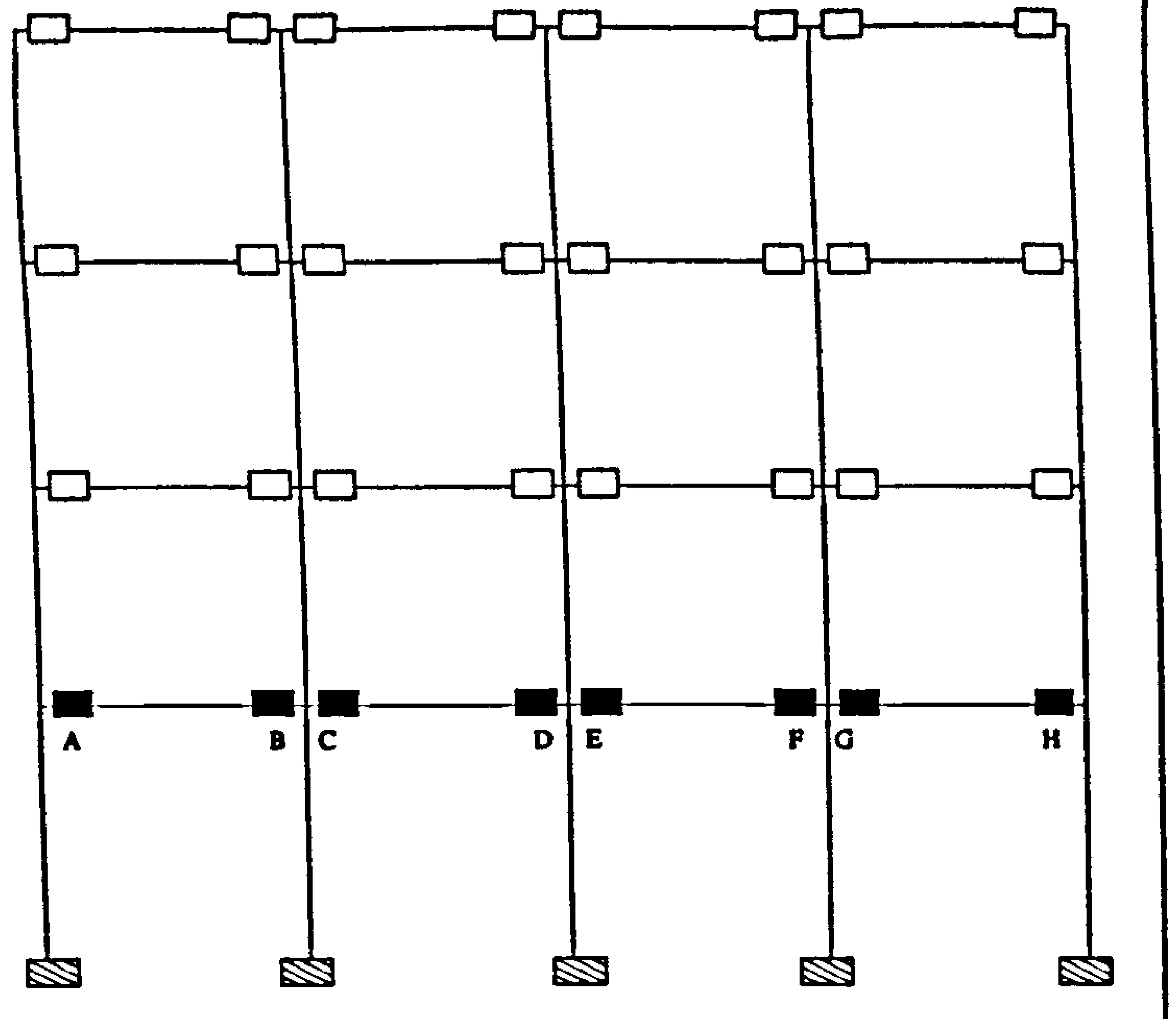
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.17	1.82
B	1.17	1.82
C	1.17	1.81
D	1.17	1.83
E	1.17	1.75
F	1.17	1.75
G	1.17	1.75
H	1.17	N/A
I	1.17	N/A
J	1.17	N/A
K	1.17	N/A
L	1.17	N/A
M	1.17	N/A
N	1.17	N/A
O	1.17	N/A
P	1.17	N/A

Key:

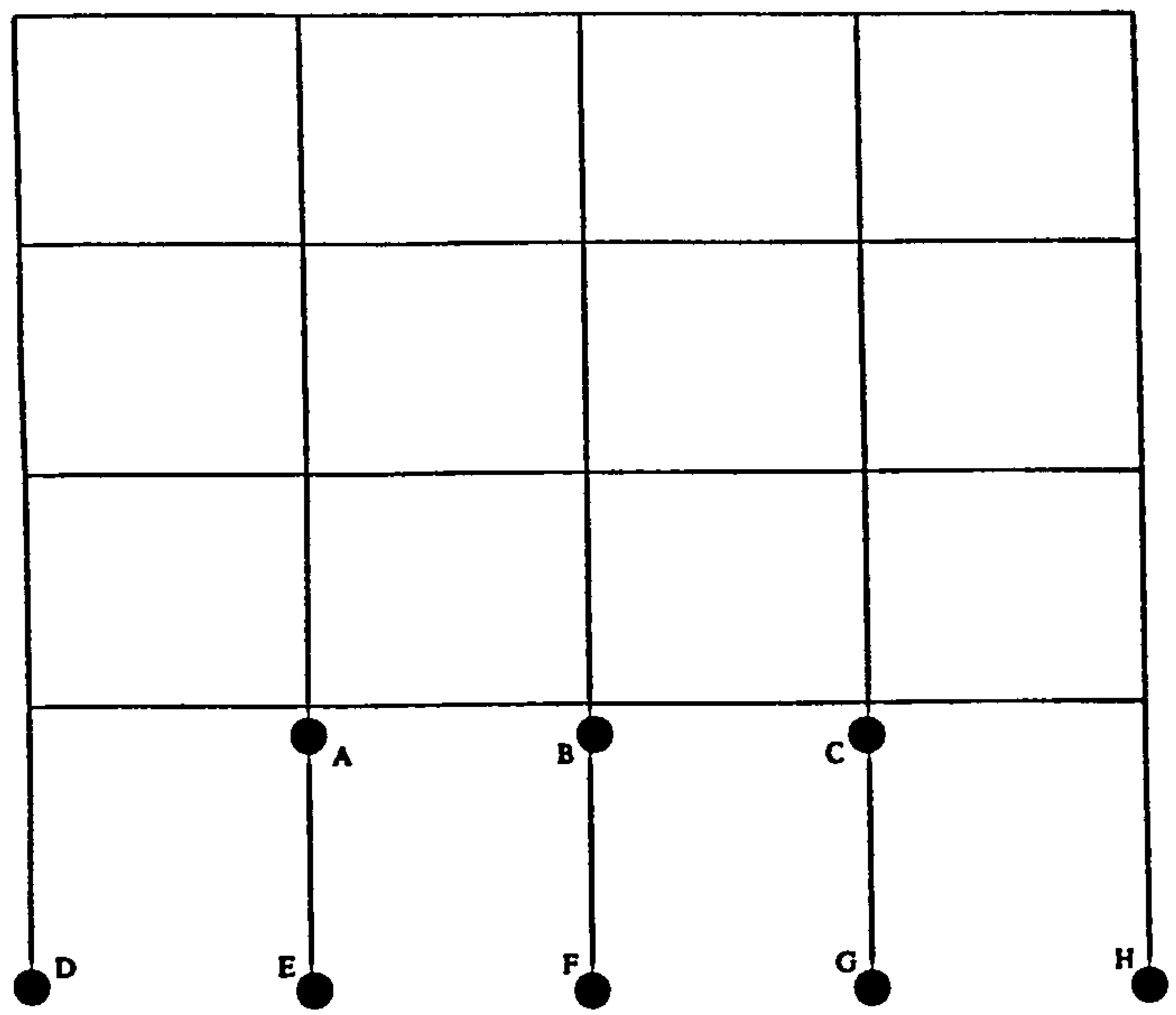
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 19
Load Case 2

Semi-Rigid Frame (Section Designation II)



Rigid Frame (Section Designation II)



Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.07	1.74
B	1.07	1.74
C	1.07	1.74
D	1.07	1.75
E	1.07	1.65
F	1.07	1.65
G	1.07	1.65
H	1.07	1.75

Key:

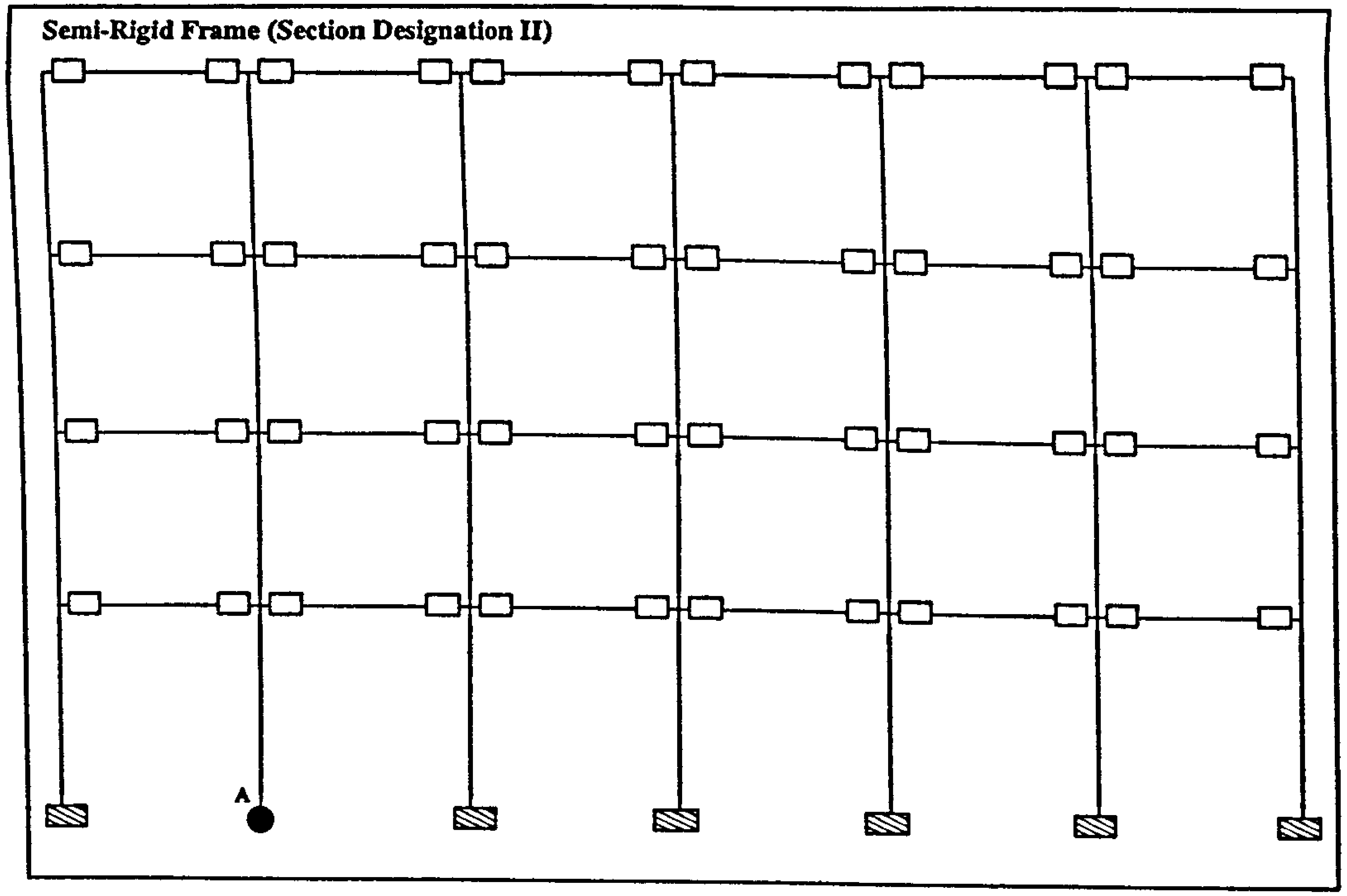
Semi-rigid connection

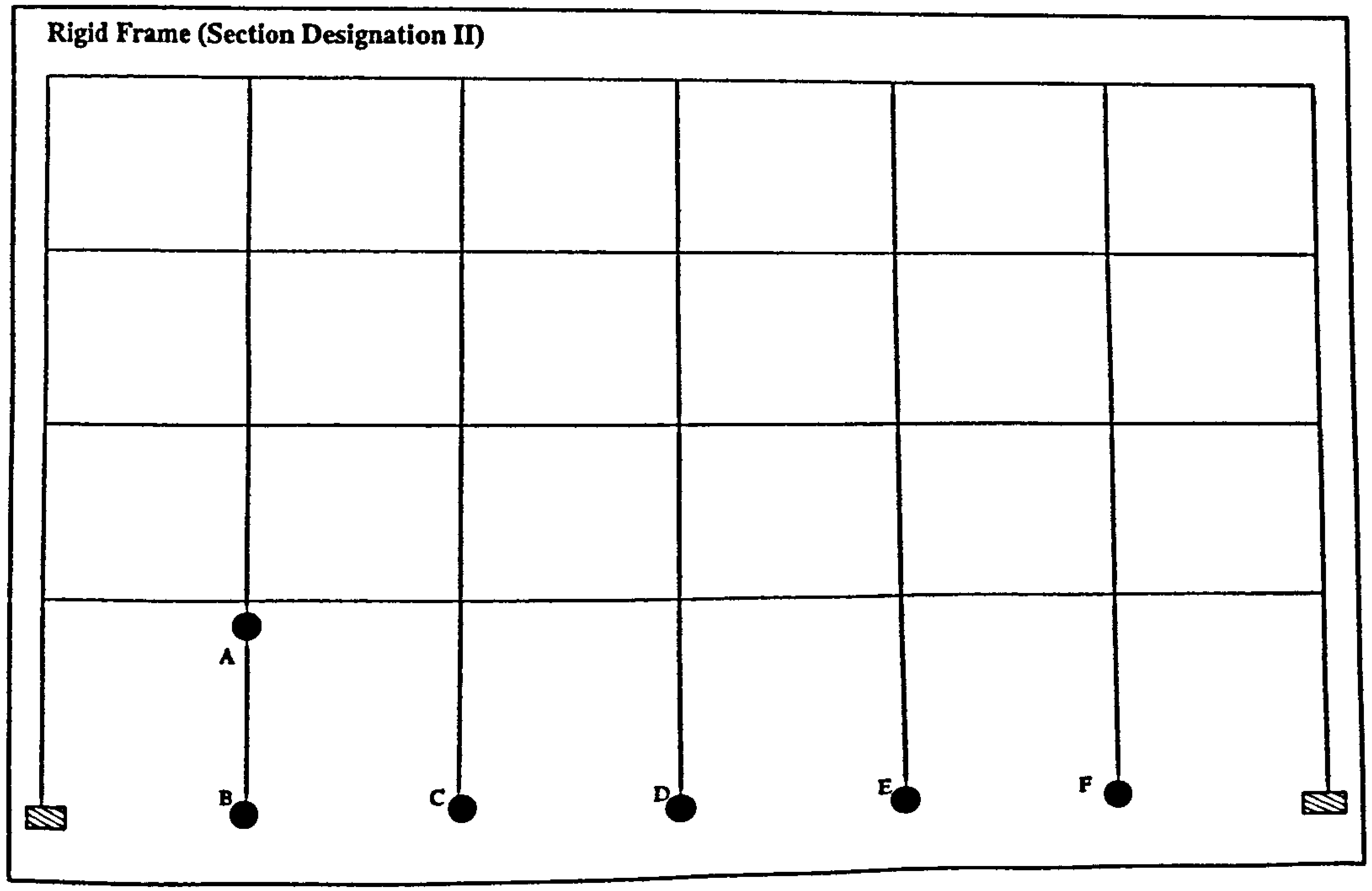
Plastification prior to ULS design load being attained

Plastification following the attainment of the design load

Member plastification

Frame 19
Load Case 3



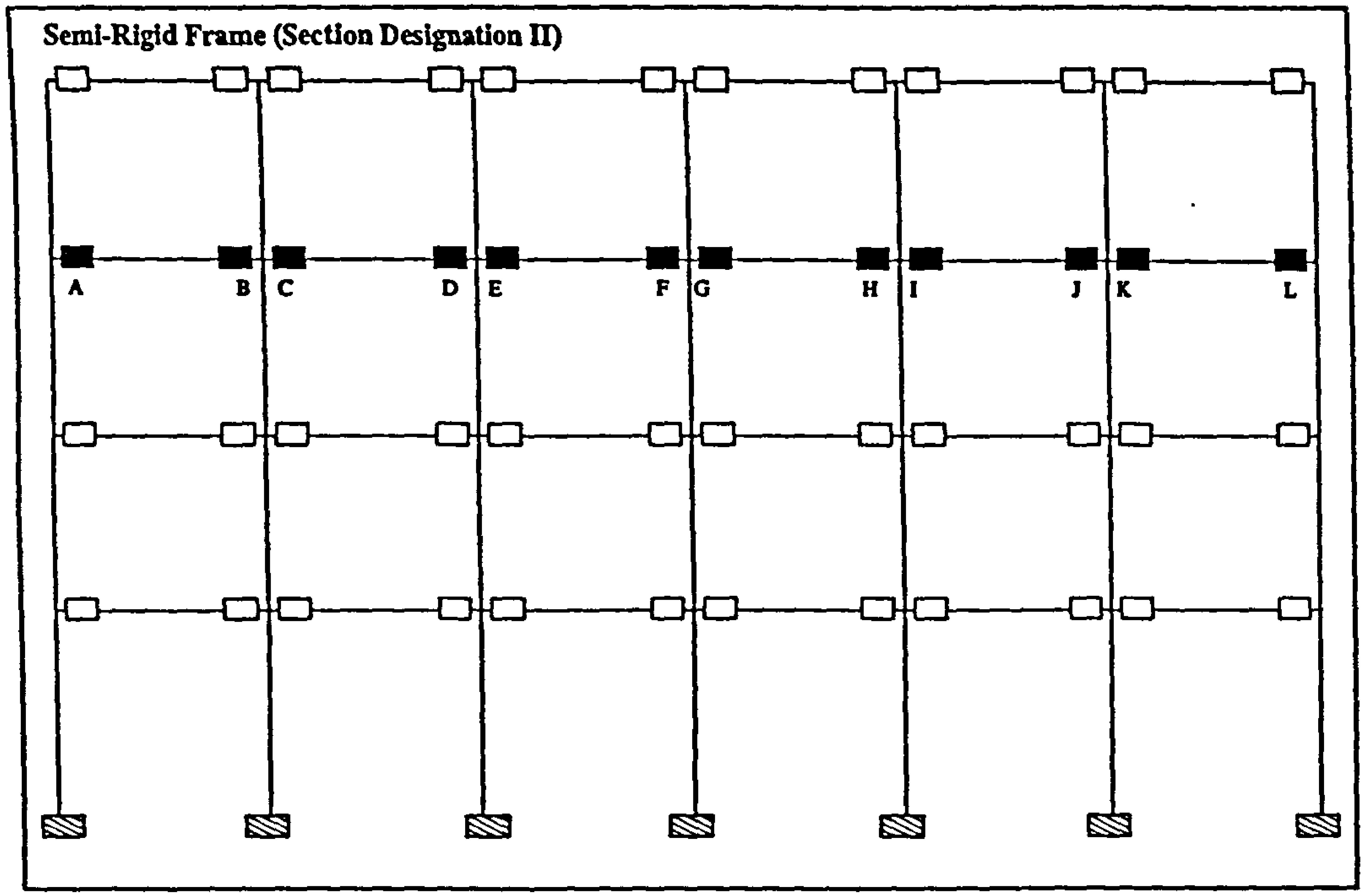


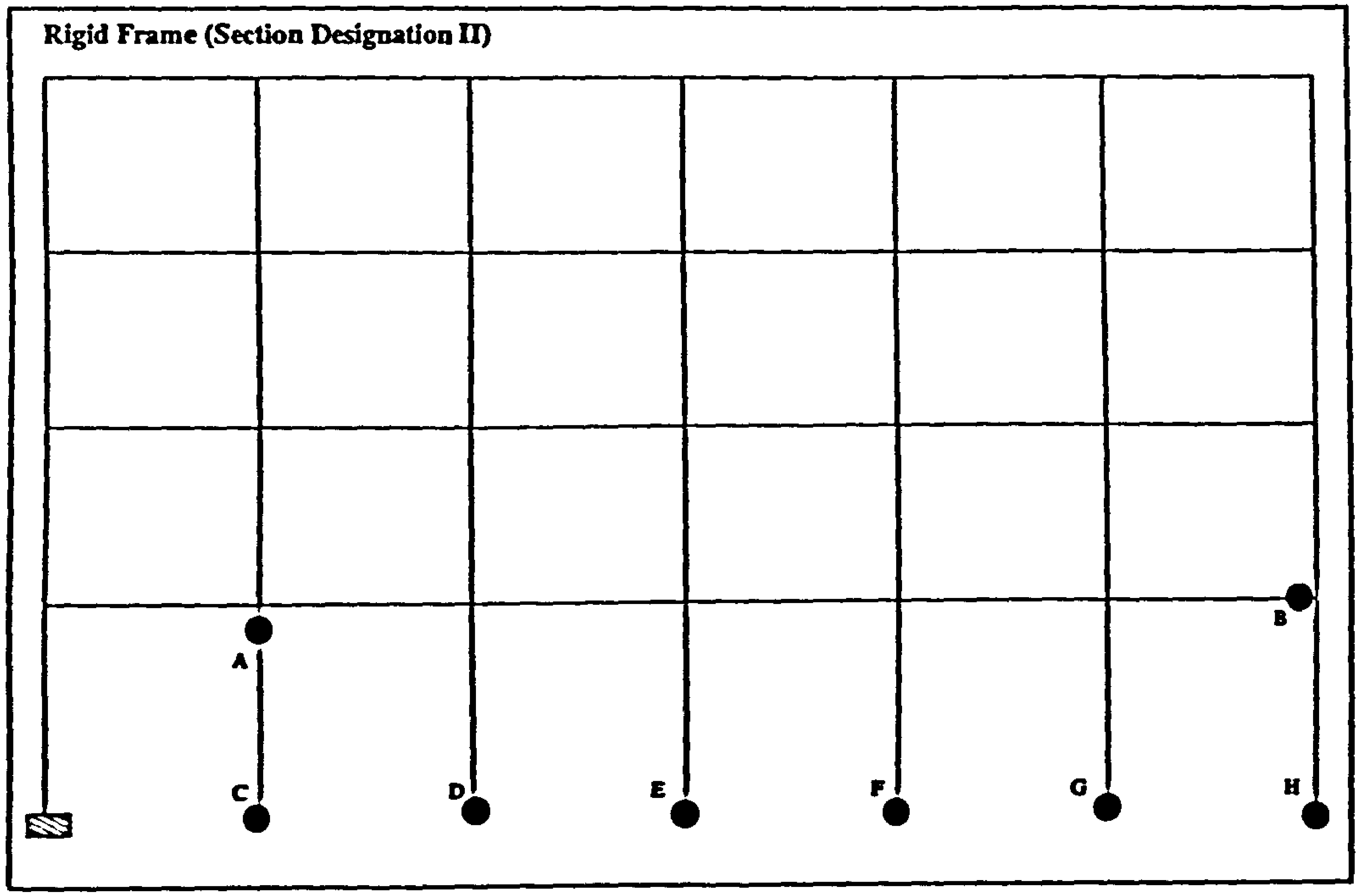
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	2.09	2.34
B	N/A	2.33
C	N/A	2.68
D	N/A	2.67
E	N/A	2.68
F	N/A	2.68

Key:
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 20
Load Case 1

Plastic-hinges in minor -axis "wind-moment" frames.



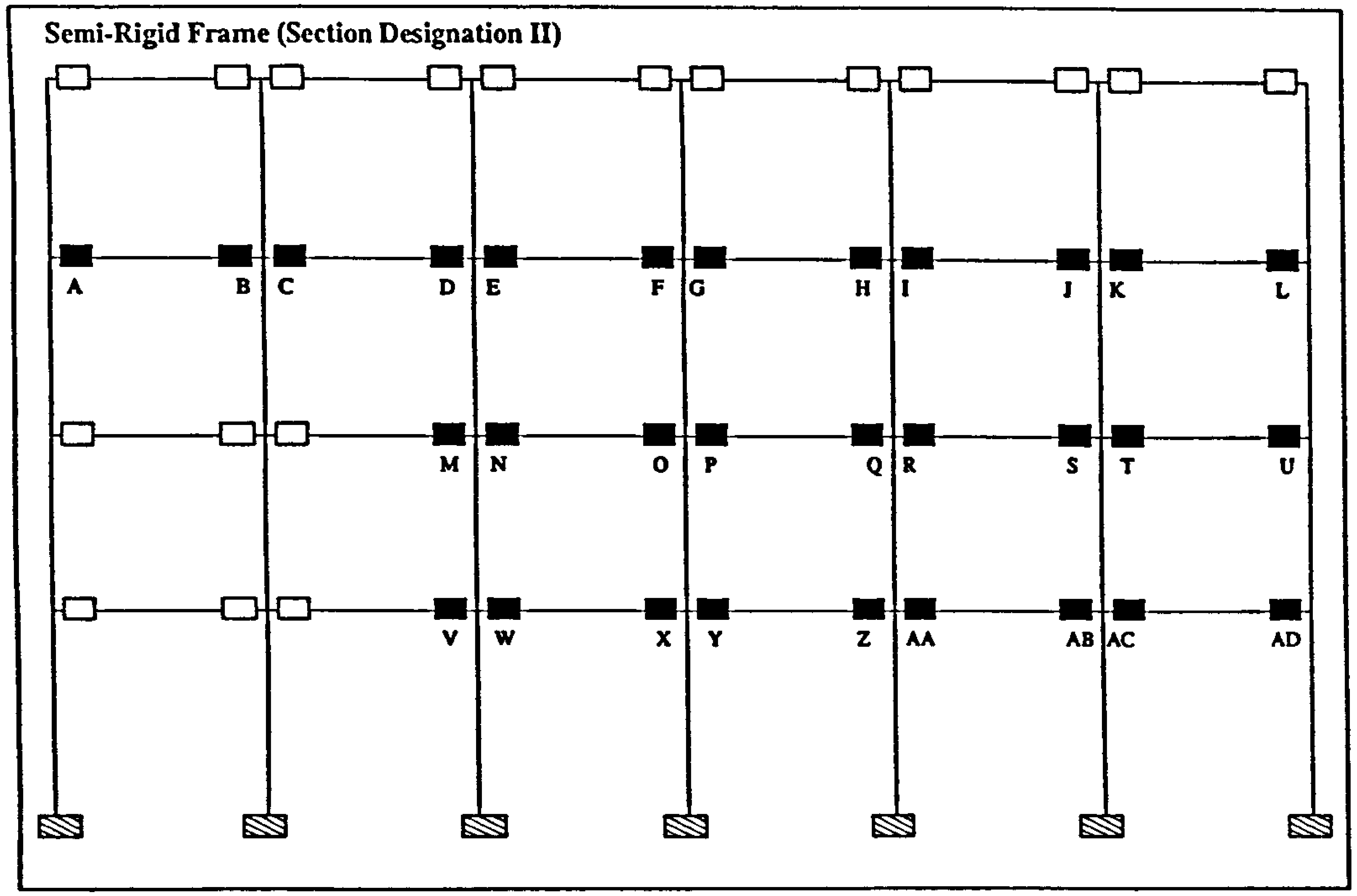


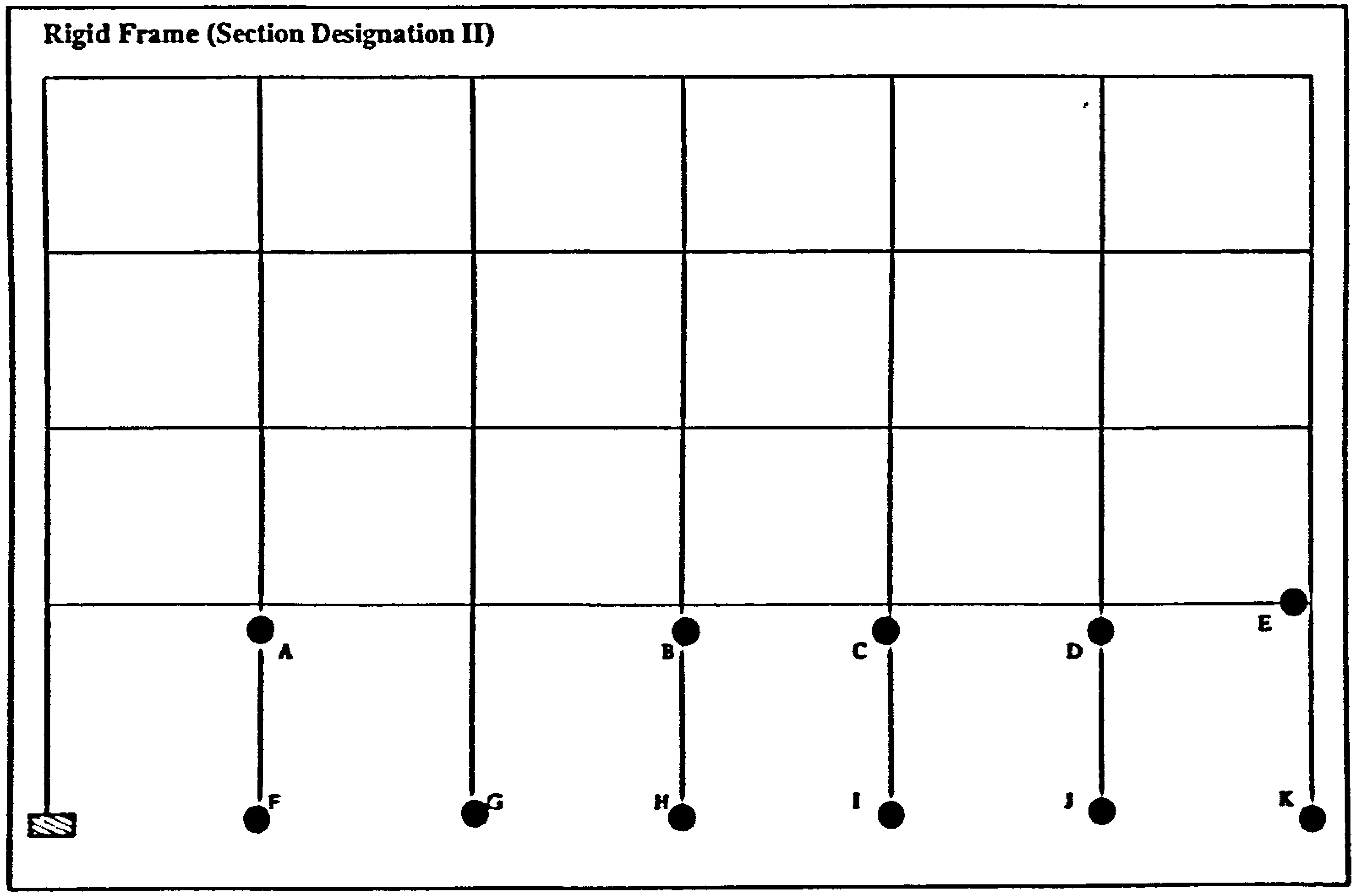
Hinge Loaction	Load Level at Hinge Formation	
	Semi-rigid	Rigid
A	1.13	1.79
B	1.13	1.78
C	1.13	1.78
D	1.13	1.77
E	1.13	1.76
F	1.13	1.77
G	1.13	1.76
H	1.13	1.80
I	1.13	N/A
J	1.13	N/A
K	1.13	N/A
L	1.13	N/A

Key:

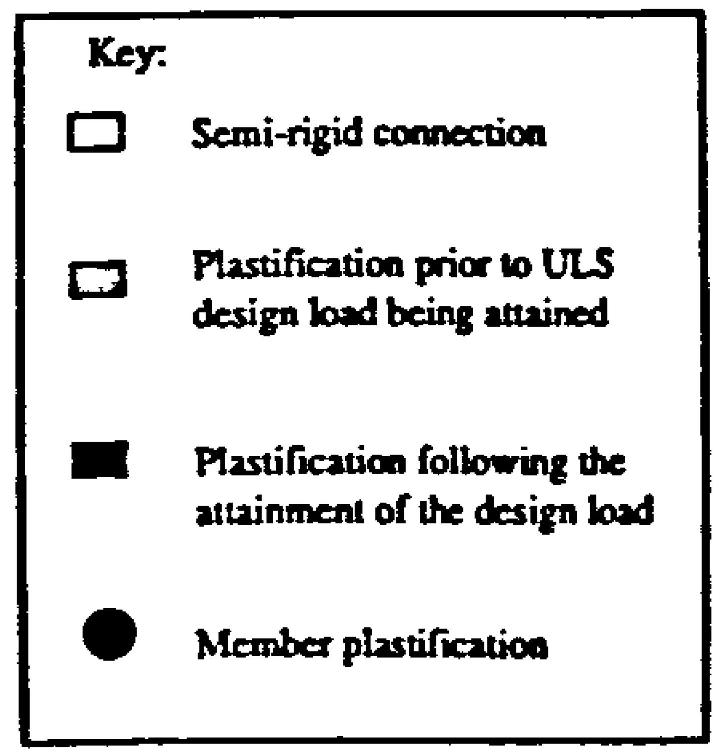
- Semi-rigid connection
- Plastification prior to ULS design load being attained
- Plastification following the attainment of the design load
- Member plastification

Frame 20
Load Case 2





Hinge Loaction	Load Level at Hinge Formation Semi-rigid	Rigid
A	1.19	1.84
B	1.19	1.85
C	1.19	1.85
D	1.19	1.84
E	1.19	1.73
F	1.19	1.80
G	1.19	1.76
H	1.19	1.76
I	1.19	1.76
J	1.19	1.75
K	1.19	1.83
L	1.19	N/A
M	1.19	N/A
N	1.19	N/A
O	1.19	N/A
P	1.19	N/A
Q	1.19	N/A
R	1.19	N/A
S	1.19	N/A
T	1.19	N/A
U	1.19	N/A
V	1.19	N/A
W	1.19	N/A
X	1.19	N/A
Y	1.19	N/A
Z	1.19	N/A
AA	1.19	N/A
AB	1.19	N/A
AC	1.19	N/A
AD	1.19	N/A



Frame 20
Load Case 3

Published work.

Two papers have been published for work presented in Chapter 2, written jointly by the author and Prof. David Anderson. The papers were entitled and published as follows:

1. Economic comparisons between simple and partial-strength design of braced steel frames, Third International Workshop on Connections in Steel Structures, University of Trento, 1995.
2. Economics of steel frames with partial-strength connections, International conference on Advances in Strategic Technologies, Universiti Kebangsaan Malaysia, Bangi, Selangor, Malaysia, 1995.

Work presented in Chapter 3 was published in the form of report to The Steel Construction Institute on April 1996, written by the author, Prof. David Anderson, and Dr. Nigel Brown. The title of the report was "Wind-moment design of unbraced frames using Eurocode 3 and standard ductile connections".

Three reports have been presented to Pneutek Inc., for work presented in Chapter 6. The reports were jointly written by the author, Dr. J.T. Mottram, and Prof. David Anderson. The reports were entitled and published as follows:

1. Push tests for composite steel-concrete beams with pin-connected shear studs, Report to Pneutek (Europe) Ltd. on Test No. 1 to 5, July 1996.
2. Push test for composite steel-concrete beams with pin-connected shear studs, Report to Pneutek (Europe) Ltd. on Test No. 6, August 1996.

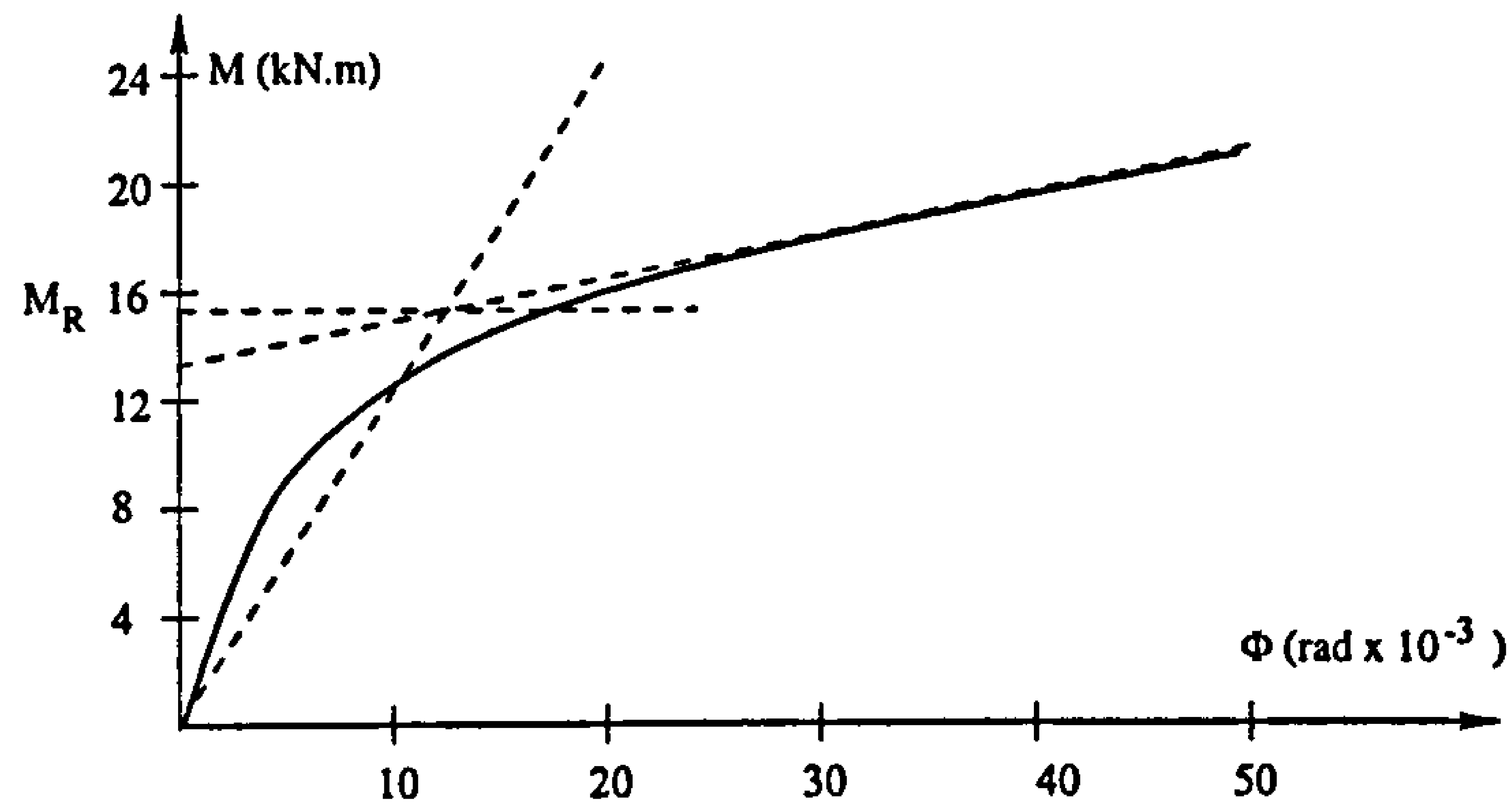


Fig. 4.2(s). Test No. 20 (M_R = 15.5 kN.m)

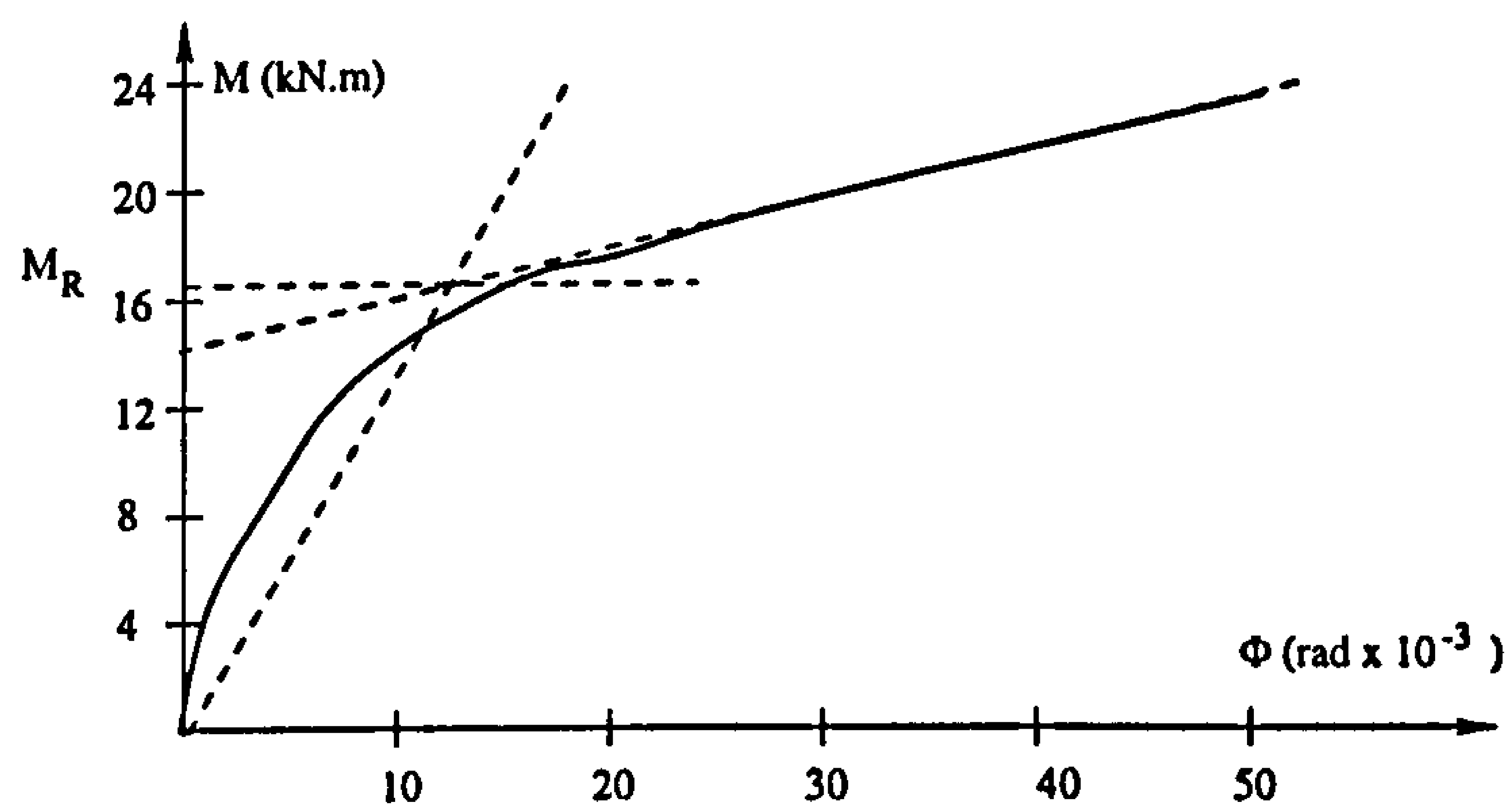


Fig. 4.2(t). Test No. 21 (M_R = 16.5 kN.m)

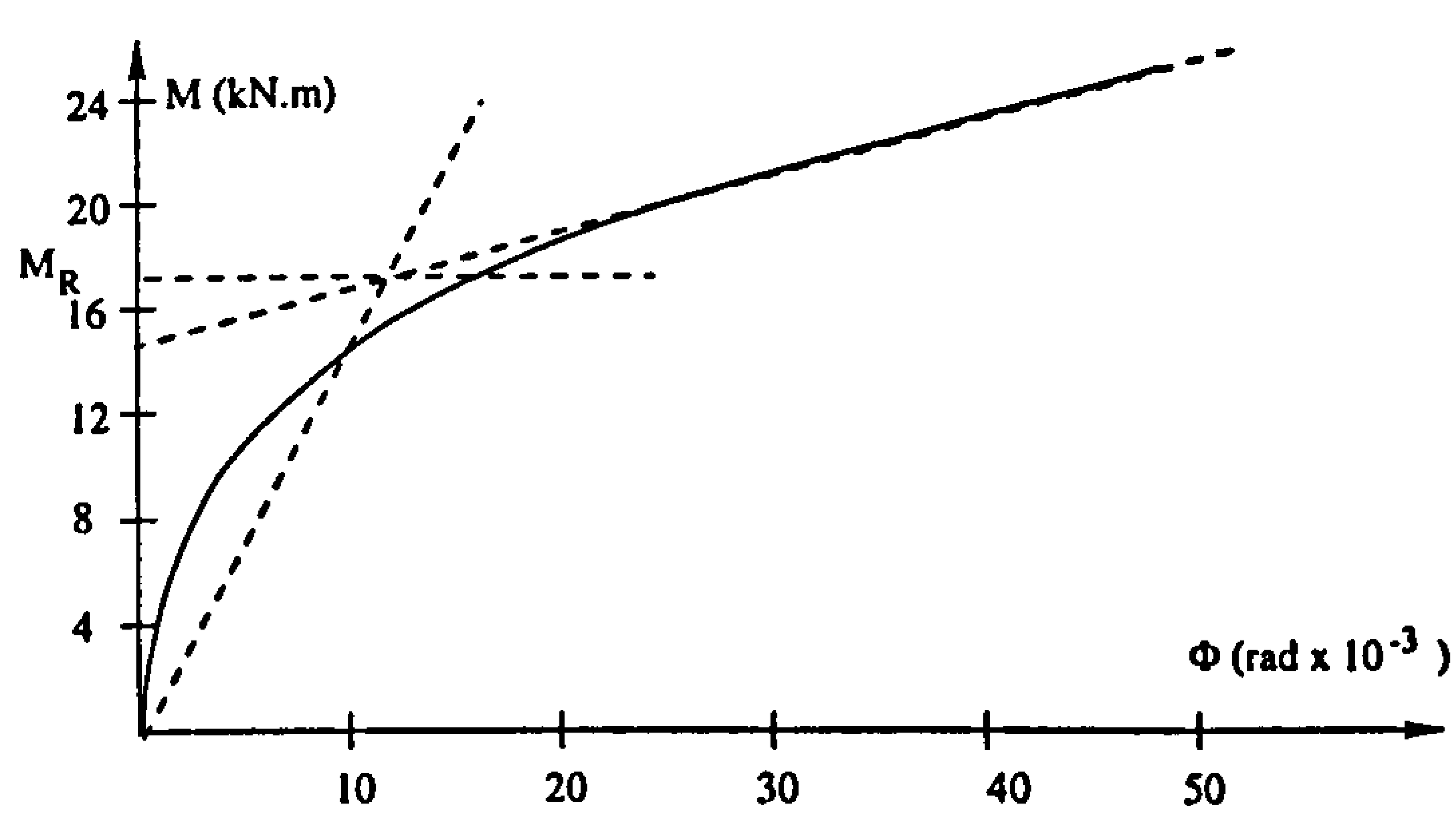


Fig. 4.2(u). Test No. 22 (M_R = 17.0 kN.m)